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HOLDING CAPACITY OF PLATE ANCHORS

By R. M. Beard

October 1980

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This handbook-style report has been prepared to assist the engineer with limited geotechnical experience, in determining the holding capacity of a plate anchor for all loading conditions – short-term static holding capacity, long-term static holding capacity, impulse loading holding capacity, cyclic loading holding capacity, cyclic creep holding capacity, and the effects of earthquake loading. A flow diagram leads the user to appropriate

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1. INTRODUCTION

Plate anchors are being used more frequently in ocean construction. Plate anchors are identified by the way they are embedded into the seafloor. Examples are propellant-embedded and vibratory-embedded anchors. The Navy currently uses propellant-embedded anchors. Their selection stems from three major advantages over conventional anchors: (1) they can efficiently resist loads in any direction, including uplift loads; (2) they can be placed at specific points; and (3) they offer a high holding capacity relative to their weight. A disadvantage of plate anchors is that they offer reduced holding capacity once their ultimate capacity has been exceeded or significant movement occurs. As a consequence, plate anchors must be designed to limit upward movement by careful selection of design loads in accordance with the types of loading expected during the anchor's life.

This report provides procedures for determining allowable design loads for plate anchors under all types of static and dynamic loading conditions in sediment seafloors. The procedures presented are necessarily simplified for broader user application. In doing so, certain soil conditions and types have been excluded to prevent the procedure from becoming overly conservative in other areas. Guidance in identifying sites with these conditions is provided, and reference is made to reports that provide additional procedures for analyzing these conditions. Specific guidance is given and procedures are given for propellant-embedded anchors.

1.1 BACKGROUND

Figure 1-1 illustrates several of the terms used in evaluating anchor holding capacity. The depth of the anchor fluke embedded in the seafloor is termed D, and the fluke width is called B. Infinite strip, square, or circular flukes are generally assumed in model studies or analyses. However, rectangular flukes can be evaluated by applying a shape factor identical to that used by Skempton (1951) for rectangular footings.

"Deep anchor failure" defines a situation in which the sediment surface is not affected when the anchor is loaded to failure. As the anchor is displaced, the soil tends to flow from above to below the anchor. "Shallow anchor failure" defines a situation in which the soil surface is bulged when the anchor is loaded to failure. As the anchor is displaced, a soil plug over the anchor is pushed out of the sediment. A term called the "relative depth of embedment" (D/B) is used to help define shallow and deep anchor behavior. This term is a function of soil type and strength, and determines which of these two modes of failure will govern when an anchor is extracted. These failure modes are illustrated in Figure 1-1.

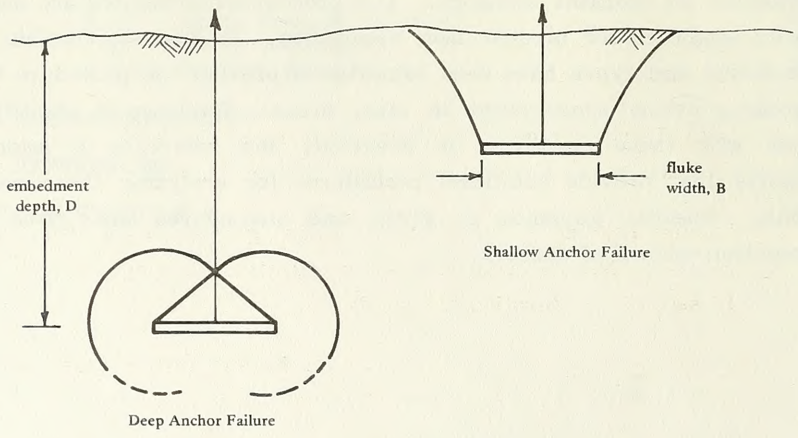

Figure 1-1. Definition of terms.

The equation used to predict anchor holding capacity (Vesic, 1969) with inclusion of a shape factor term (Skempton, 1951) and patterned after bearing capacity equations for footings is as follows:

$$F = A(c\,\bar{N}_c + \gamma_b\, D\, \bar{N}_q)(0.84 + 0.16\, B/L) \tag{1.1}$$

where
F = holding capacity
A = fluke area
c = soil cohesion
γ_b = soil buoyant unit weight
D = fluke embedment depth
B = fluke width or diameter
L = fluke length or diameter
$\bar{N}_c$, $\bar{N}_q$ = holding capacity factors

This is a general equation that can be applied to a variety of embedment, soil, and loading conditions.

The design of plate anchors can be separated into two general loading conditions: static and dynamic. Static loading includes both short-term loading and long-term loading. Dynamic loading includes impulse loading, cyclic loading, and earthquake loading. Each can be defined as follows:

- Short-Term Loading - An increasing load to failure, such that in fine-grained soils drainage does not occur.
- Long-Term Loading - A fairly uniform static load of sufficient duration that full drainage occurs in fine-grained soils.
- Impulse Loading - Nonrhythmic loads greater than the static capacity, less than 10 minutes in duration for clays or less than 10 seconds in duration for sands.
- Cyclic Loading - A repetitive anchor loading with a double amplitude magnitude greater than 5% of the static capacity.

- Earthquake Loading - A cyclic loading induced to the entire soil mass by earthquake energy.

1.2 CEL PROPELLANT-EMBEDDED ANCHORS

The Navy's interest in plate anchors is concentrated on propellant-embedded anchors of which four are available for use. Designated by their nominal holding capacity in kips, they are the 10K, 20K, 100K, and 300K propellant-embedded anchors. Specific guidance concerning these anchors is given in this report. The anchors appear and function similarly. Figure 1-2 shows the CEL 20K propellant-embedded anchor, and a functional sequence is shown in Figure 1-3. On contacting the seafloor, the touchdown probe triggers the safe/arm device which in turn initiates the propellant contained in the gun barrel. The propellant burn expels the fluke from the anchor at high velocity, while reaction is provided by the gun barrel and reaction vessel. The fluke then penetrates the seafloor, dragging successive loops of downhaul cable behind it. The fluke penetrates a distance, D_p, in an edge-on orientation to the seafloor and is keyed (see Figure 1-3) to present a large area bearing against the sediments to resist pullout.

1.3 RELATED REPORTS

This handbook-style report is based on a number of reports that provide more specific detail on the various subjects of the procedure presented herein. There are also associated reports on anchor hardware, penetration, and site survey that provide additional details.

1.3.1 Anchor Holding Capacity

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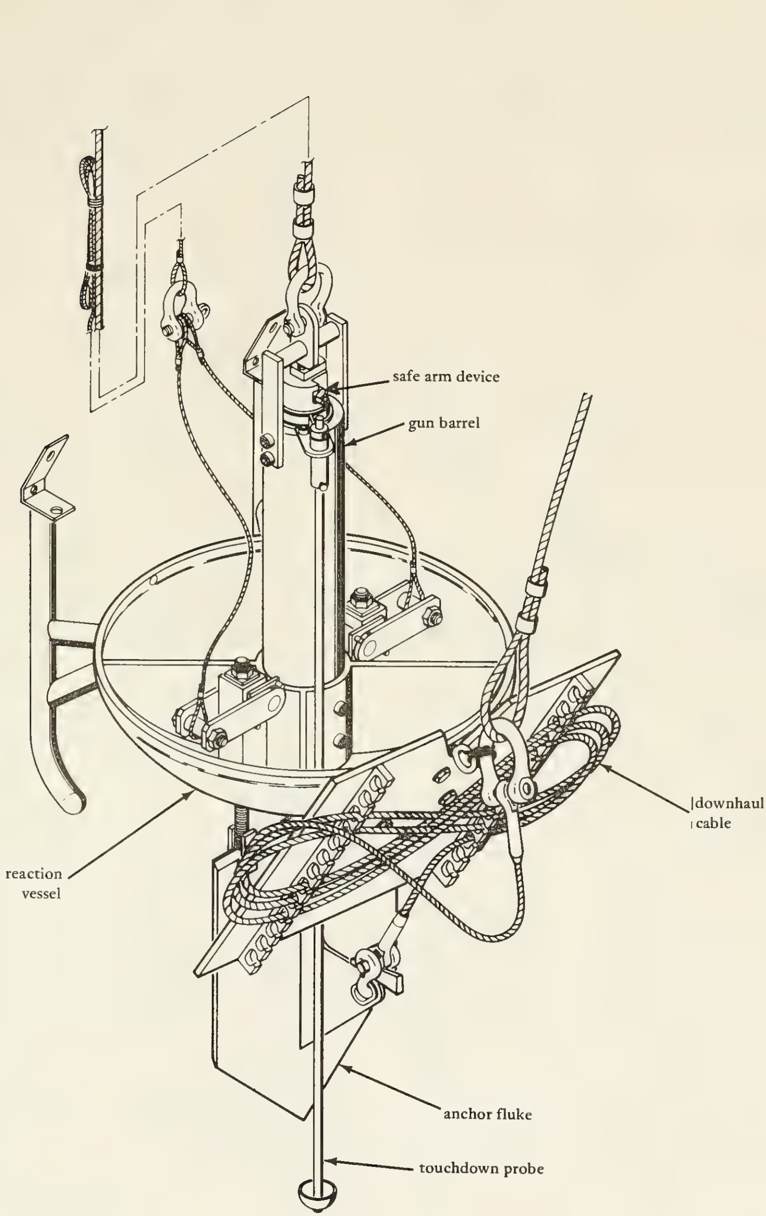

Figure 1-2. CEL 20K propellant-embedded marine anchor.

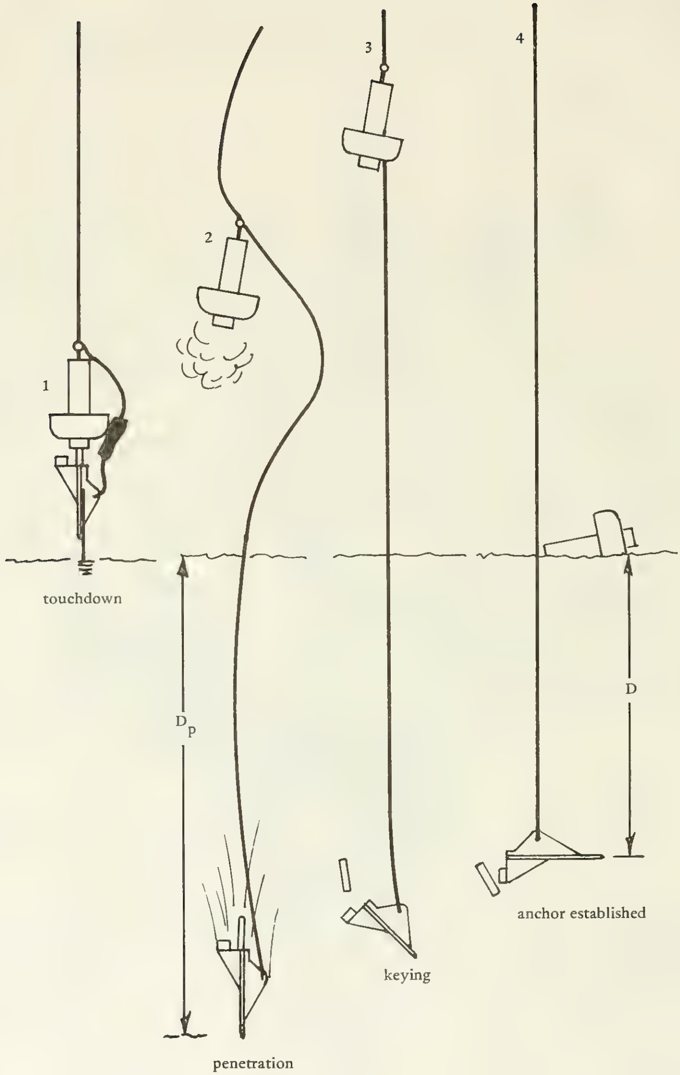

Figure 1-3. Penetration and keying of a propellant-embedded anchor.

Douglas, B. J. (1978). Effects of rapid loading rates on the holding capacity of direct embedment anchors, Civil Engineering Laboratory, PO No. M-R420. Port Hueneme, Calif., Oct 1978.

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Kulhawy, F. H., D. A. Sangrey, and S. P. Clemence (1978). Direct embedment anchors on sloping seafloors state-of-the-art, Civil Engineering Laboratory, P.O. Report M-R510. Port Hueneme, Calif., Oct 1978.

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1.3.2 Site Survey

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Naval Facilities Engineering Command (1971). Soil mechanics, foundations, and earth structures, Design Manual DM-7. Washington, D.C.

1.3.3 Penetration

True, D. G. (1975). Penetration of projectiles into seafloor soils, Civil Engineering Laboratory, Technical Report R-822. Port Hueneme, Calif., May 1975.

1.3.4 Anchor Siting and Verification

Malloy, R. J., and P. J. Valent (1978). Acoustic siting and verification of the holding capacity of embedment anchors, Civil Engineering Laboratory, Technical Note N-1523. Port Hueneme, Calif., Jul 1978.

1.3.5 Anchors

Babineau, P. H., and R. J. Taylor (1976). CEL 10K propellant-actuated anchor operations manual, Civil Engineering Laboratory, Technical Memorandum M-42-76-3. Port Hueneme, Calif., Apr 1976.

Babineau, P. H., and D. G. True (1978). CEL 100K deep water propellant-actuated embedment anchor system operations manual, Civil Engineering Laboratory, Technical Memorandum M-42-78-15. Port Hueneme, Calif., Nov 1978.

Beard, R. M. (1973). Direct embedment vibratory anchor, Naval Civil Engineering Laboratory, Technical Report R-791. Port Hueneme, Calif., Jun 1973.

Naval Facilities Engineering Command (1979a). 20K propellant-embedded marine anchor, Technical Manual P-3-20K (preliminary). Washington, D.C., Jul 1979. (FOUO)

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Taylor, R. J. (1976). CEL 20K propellant-actuated anchor, Civil Engineering Laboratory, Technical Report R-837. Port Hueneme, Calif., Mar 1976.

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1.3.6 Sediments

Demars, K. R., and R. J. Taylor (1971). Naval seafloor sampling and in-place equipment: A performance evaluation, Naval Civil Engineering Laboratory, Technical Report R-730. Port Hueneme, Calif., Jun 1971.

Lee, H. J. (1973a). In-situ strength of seafloor soil determined from tests on partially disturbed cores, Naval Civil Engineering Laboratory, Technical Note N-1295. Port Hueneme, Calif., Aug 1973.

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Lee, H. J. (1973c). Engineering properties of some North Pacific and Bering Sea soils, Naval Civil Engineering Laboratory, Technical Note N-1283. Port Hueneme, Calif., Aug 1973.

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Lee, H. J. (1978). Physical properties of biogenous sediments from the eastern equatorial Pacific Ocean, Civil Engineering Laboratory, Technical Memorandum M-42-78-4. Port Hueneme, Calif., Feb 1978.

Rocker, K. (1974). "Vane shear strength measurements on Leg 27 sediment," Initial Reports of the Deep Sea Drilling Project, Volume 27. Washington, D.C., 1974.

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1.4 GUIDE TO THIS DOCUMENT

This report covers a broad range of topics dealing with the holding capacity of plate anchors. As an aid to the user, Figure 1-4 presents the steps necessary to determine the applicable holding capacity of an anchorage. The items in Figure 1-4 are referenced to appropriate sections of the report. Briefly these sections are:

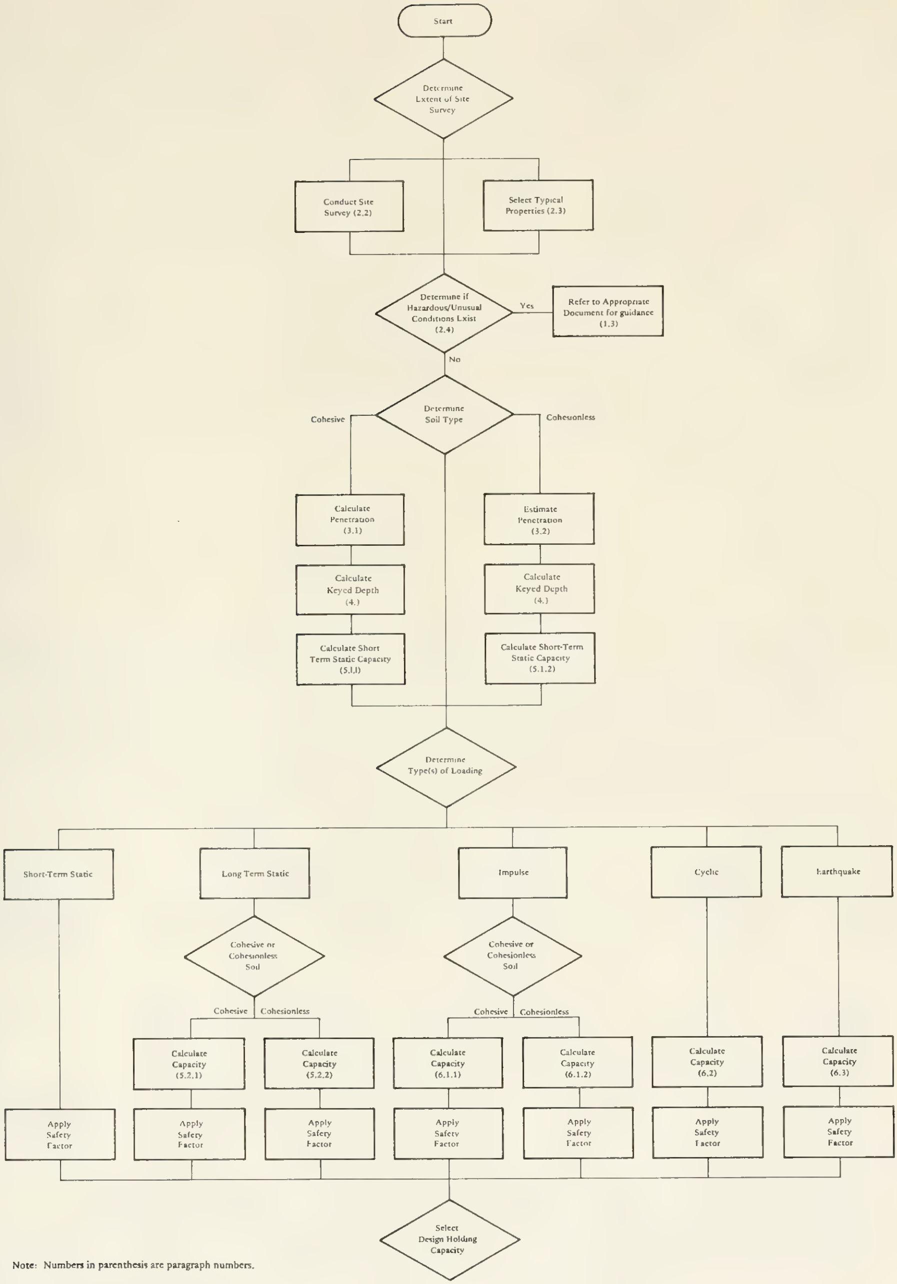
- Section 2, SITE SURVEY, provides recommended site surveys and typical properties.

- Section 3, PENETRATION, presents methods for estimating penetration in cohesive and cohesionless soils.

- Section 4, FLUKE KEYING, discusses fluke keying distances in cohesive and cohesionless soils.

- Section 5, STATIC HOLDING CAPACITY, presents methods for calculating holding capacity under short-term and long-term static loading.

- Section 6, DYNAMIC HOLDING CAPACITY, presents methods for estimating holding capacity under impulse, cyclic, and earthquake loading.



Note: Numbers in parenthesis are paragraph numbers.

Figure 1-4. Flow diagram for predicting holding capacity.

2. SITE SURVEY

This section lists required site data for designing plate anchors and presents recommended site surveys, gives typical properties to be used when site surveys are limited or cannot be conducted, and lists hazardous/unusual conditions that create additional problems for the designer.

2.1 SITE-SURVEY REQUIREMENTS

Data requirements differ according to the type of analysis and soil expected. A matrix of site data and analysis types is given in Table 2-1. To use Table 2-1, find the analysis type column and look down that column for the soil type (1 indicating cohesive soil, and 2 indicating cohesionless soil). Each time the proper soil type is encountered, look to the left-hand column to find the soil data requirement. For example, short-term static loading in a cohesionless soil requires data on the friction angle (ϕ), density, and grain size distribution. In general, the variation of the properties with depth is required. Brief comments about the data requirements are as follows:

(a) Soil Strength - For cohesive soils the undrained shear strength, sensitivity, and the drained friction angle should be known. For cohesionless soils the friction angle is required.

(b) Soil Density - Required for both cohesive and cohesionless soils. For cohesionless soils, a descriptive relative density (loose, medium dense, dense) is required for liquefaction analysis.

Table 2-1. Required Site Data Versus Type of Analysis

Soil Data	Type of Soil[a] Under --					
	Penetration	Static Loading		Dynamic Loading		
		Short-Term	Long-Term	Impulse	Cyclic	Earthquake
Strength						
s_u	1	2				
S_t	1			1		
ϕ	2	2	1,2			
c			1			
Density	1,2	2	1,2		2	2
Plasticity					1	
Grain size	1,2	1,2	1,2	1,2	1,2	1,2
Permeability					1,2	
Origin					1,2	
Depth to rock	1,2					

[a]1 = cohesive
2 = cohesionless

(c) Soil Plasticity - Plastic Index (PI) should be determined from laboratory tests.

(d) Soil Grain Size Classification - Classification by grain size and plasticity between cohesive and cohesionless is mandatory. Classification by the Unified Soil Classification System is preferable.

(e) Permeability - Required when cyclic load capacity is to be determined.

(f) Soil Origin - Deep ocean soils should be classified by origin on the basis of dry weight percentage that is biogenic (calcareous or siliceous).

(g) Depth to Rock - Required to determine adequacy of soil cover.

A variety of tools can be used to acquire these data. They include corers, grab samplers, sub-bottom profilers, dynamic penetrometers, static cone penetrometers, in-situ vane shear devices, and drilling. Record data are another means for gathering site data.

2.2 RECOMMENDED SITE SURVEY

The type of site survey that can be conducted to gather the required data is primarily a function of time and money. For some projects perhaps a full suite of data can be gathered, while for other projects a review of record data is all that is possible. The recommended surveys hopefully strike a balance between these two extremes.

2.2.1 Deep Water

Record data should be searched to determine as much as possible about the site or surrounding area. The site evaluation should be performed sufficiently close to the expected anchor installation location to provide reliable information and samples of the soils to be encountered. Table 2-2 gives approximate survey depth requirements for each of the four CEL propellant-embedded anchors for several general soil types.

Table 2-2. Approximate Survey Depth Requirements for CEL Propellant-Embedded Anchors

Soil Type	Survey Depth, m (ft) for --			
	10K Anchor	20K Anchor	100K Anchor	300K Anchor
Soft clay	9 (30)	12 (40)	15 (50)	20 (65)
Medium clay	6 (20)	9 (30)	12 (40)	15 (50)
Loose sand	5 (15)	6 (20)	8 (25)	9 (30)
Dense sand	3 (10)	5 (15)	6 (20)	8 (25)

(a) Sub-bottom profile (3.5-kHz acoustic source) to locate bedrock, sediment, and layering, and to assess areal uniformity.

(b) A relatively undisturbed core sample at least as long as the expected depth of anchor penetration to make measurements of undrained shear strength and sensitivity (with a minivane), density, plasticity, grain size, and origin. For most soft cohesive soils, the friction angle can be estimated as 35 degrees (from CEL experience) as static short-term capacity will govern over static long-term capacity. For exceptions, measure the friction angle and the cohesion of the soil with triaxial tests at the depth of interest. When a long core is not available, analyze a sample of the upper 6 to 8 feet of soil as suggested for a longer core, then extrapolate the data and compare the extrapolation to undrained shear strength data from a Doppler penetrometer or similar device providing comparable data and depths of penetration.

There will be few cases where cohesionless sediments will be found in deep water. The recommended survey for these cases would have begun with a search of record data followed by:

(a) Sub-bottom profiling to locate bedrock and layering and assess areal uniformity.

(b) A Doppler penetrometer test to judge the relative soil strength.

(c) A sample of even limited depth for grain size and origin determination.

Determining the friction angle will be difficult, as representative samples cannot be obtained and the penetrometer test will not provide it. However, safe estimates of the friction angle and the submerged density can be made using Table 2-3.

Table 2-3. Estimated Values for Cohesionless Soil Properties

Soil Description	Friction Angle, ϕ (deg)	Buoyant Unit Weight, kg/m^3 (lb/ft^3)
Sandy silt	20	880 (55)
Silty sand	25	880 (55)
Uniform sand	30	880 (55)
Well-graded sand	35	960 (60)

2.2.2 Shallow Water

At shallow water sites, it may be less expensive and technically preferable to use adaptations of terrestrial survey techniques that provide data similar to those recommended for deep water surveys. In shallow water sites, cohesionless soils will be more common, and the use of the Standard Penetration Test will allow the determination of the friction angle. When adaptations of terrestrial survey techniques cannot be used, it is recommended that a shallow water survey follow along the lines of the recommended deep water survey.

2.3 RECOMMENDED PROCEDURES WHEN SITE SURVEY IS LIMITED

Because the seafloor is primarily a depositional rather than an erosional environment, more uniformity of sediments and sediment properties can be found there than would be found on land. Properties based on known environmental conditions can often be estimated accurately enough for site selection and preliminary design. Even in more complex areas where a site survey is definitely required, an estimate of properties to be encountered will aid in designing the survey and influencing initial thinking about the facility. Lee and Clausner (1979) have procedures for estimating the necessary properties, and those procedures are presented herein.

To estimate properties with some reliability, the designer must know some marine sedimentology. Fortunately, the basic concepts are simple. First, one must determine whether the sediments are land-derived (terrigenous) or ocean-derived (pelagic). Figure 2-1 gives an overall view of the ocean sediment distribution throughout the world.

2.3.1 Near-Shore Areas

One may assume that all continental shelves and slopes are terrigenous; also, virtually all seafloor features labeled "abyssal plains" have basically terrigenous components.* In a few areas of the world (North Atlantic or the far Northwest Pacific), other significant terrigenous deposits may well be found beyond the continental slope as a result of being downwind from major deserts. An engineer working in these areas should consult an expert from a nearby oceanographic institution for local information, the literature of marine geology, and ocean engineering research institutions, such as the Civil Engineering Laboratory. This consultation should be for all areas and for all initial searches by ocean engineers with little background in geotechnology.

2.3.2 Deep Ocean Areas

The sediments of the deep ocean basins far from land are determined by two factors: (1) sea surface biological productivity and (2) dissolution of calcium carbonate. Where productivity is high (such as the northern Pacific near the Aleutians, equatorial Pacific, and the region surrounding Antarctica), one finds siliceous ooze, a sediment composed of the remains of organisms whose hard parts are opaline silica. In those areas where calcium carbonate dissolution is less than the carbonate supply, calcareous ooze (a sediment composed of the remains of organisms whose hard parts are calcium carbonate) may be found. At water depths shallower than the calcite compensation depth (CCD), calcareous sediments are almost always found. The sediment is defined as a calcareous ooze if its calcium carbonate content is more

*These were probably brought down by turbidity currents.

than 30%; i.e., if it is not significantly diluted by terrigenous or siliceous materials. Generally, dilution by other materials is significant only near shore; on abyssal plains; and in the high productivity, siliceous ooze areas. The CCD has been mapped on a worldwide basis and is shown in Figure 2-2; one can determine whether calcareous ooze may be found by comparing the actual water depth with the CCD. Calcareous ooze typically becomes more coarse as water depth decreases (Figure 2-3). The equatorial Pacific typically has alternating bands of siliceous and calcareous ooze. Where biogenic (calcareous and siliceous) oozes are not found, one finds pelagic clay, an extremely slowly sedimented material composed primarily of wind-blown dust.

2.3.3 Terrigenous Sediments

Terrigenous sediments are the most complex and varied of the sediment types. The typical terrigenous material is probably a slightly plastic clayey silt; however, vast sand beds and some plastic clay deposits also exist. Layered deposits of sand, silt, and clay are common. Sedimentation rules are difficult to define. In a stable environment, grain size would decrease with distance from shore; however, since dynamic processes are always active, this often does not occur. If the sea level is rising (e.g., off the east coast of the United States), one can almost assume that grain size will become finer near shore. For any particular location, an expert is probably available to estimate the types of sediments that can be expected. The charts of the National Ocean Survey* also provide estimates of sediment type, although some of the classifications do not relate very well to engineering application (e.g., "brown mud"). However, the split between sand and clay or silt ("mud") appears reliable.

Sediment classification on the basis of grain size or origin may not be adequate for engineering application. The design engineer should also know something about the state of the sediment. For cohesive sediments, three terms are important: (1) overconsolidated, (2) normally consolidated, and (3) underconsolidated.

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*Formerly U.S. Coast and Geodetic Survey, U.S. Department of Commerce.

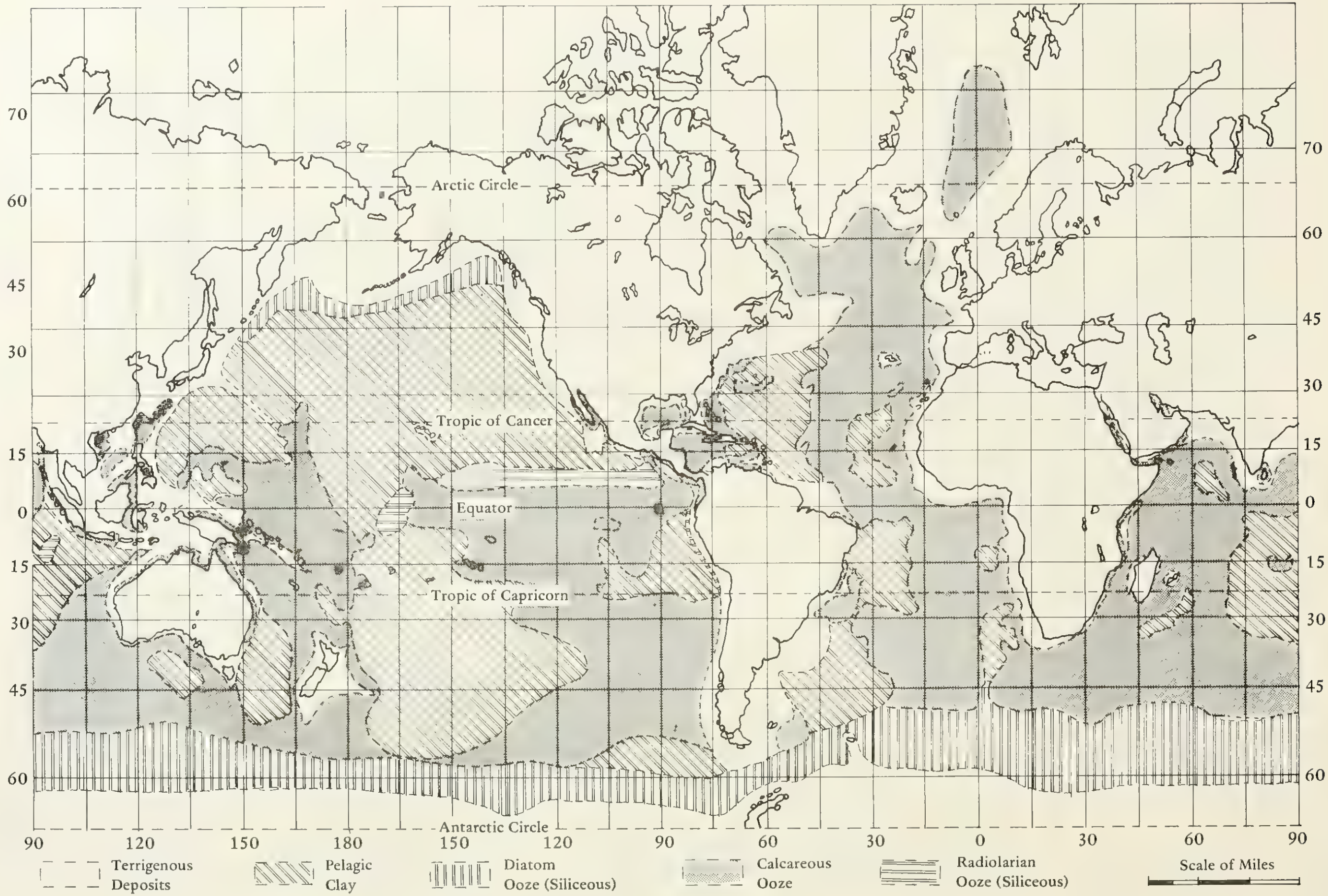

Figure 2-1. Ocean sediment distribution (Herrmann et al., 1972).

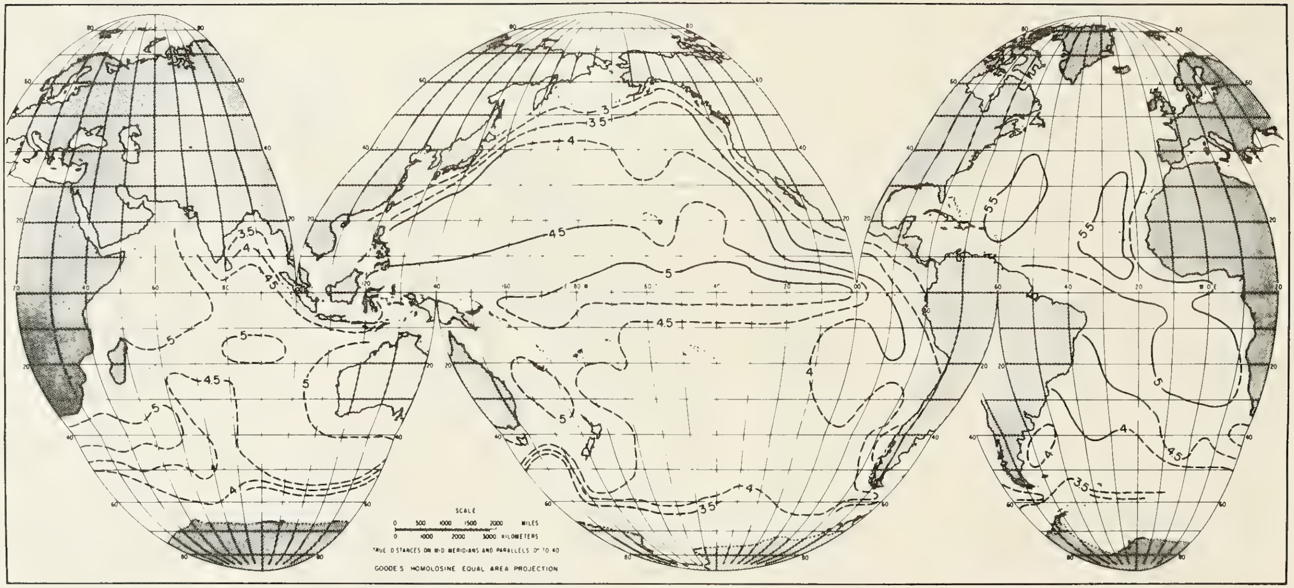

Figure 2-2. Topography of the calcite compensation depth (CCD). Calcareous sediments are found only in those locations where actual water depth is less than the CCD; numbers on contours denote kilometers below sea surface (from Berger and Winterer, 1974).

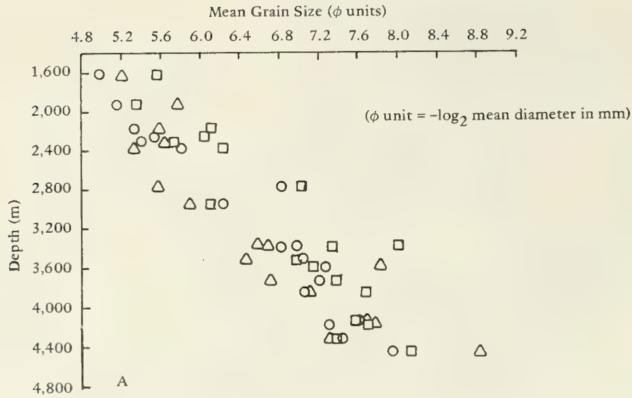

Figure 2-3. Mean grain size of carbonate sediment samples versus depth of deposition (from Johnson, Hamilton, and Berger, 1977).

Normally consolidated sediments are the rule in the deep ocean; there are materials that have never been loaded by overlying material more than they are now. Overconsolidated sediments have had a greater load (overburden) in the past and have since lost it by chemical processes or mechanical erosion. Underconsolidated sediments are young and have not come to equilibrium with the weight of overlying material.

If one assumes all deep ocean sediments to be normally consolidated, one will usually be correct and conservative. A few important exceptions do exist, but these need not concern the engineer unless finding an unusually strong (overconsolidated) sediment would lead to a less conservative design.

Much of the near shore is overconsolidated. Since this is usually a desirable situation and since it is so common, it would be valuable to find overconsolidated locations and determine their overconsolidation level.* There are no fast rules for locating overconsolidated sediments except that exposed locations (tops of rises, passages) are more likely to be overconsolidated than are protected locations (basins).

*Overconsolidation ratio (OCR).

Underconsolidated sediments are almost always found in active river deltas such as that of the Mississippi River. If deposition is fast enough, there may be almost no buildup of strength with subbottom depth. If one is operating near the mouth of a river, such as the Mississippi, Amazon, or Nile, one should be prepared for unusually weak sediments and never rely on typical property profiles. Other areas of underconsolidated sediments include embayments that exhibit high depositional characteristics with low water velocity.

2.3.4 Sediment Property Selection

With this brief background as a basis, it is recommended that the designer use the following procedure for selecting typical sediment strength, density, and sensitivity data.

(1) If the site is on the continental shelf or slope, the sediment is assumed terrigenous. Available National Ocean Survey charts are consulted to determine whether the sediment is primarily sandy or cohesive ("mud"). If the sediment is cohesive, Figure 2-4, which gives a lower bound for the strength distribution for a normally consolidated sediment, is referred to. A search for strong indications of overconsolidation is made: recorded outcrops of older sediments, exposed location (rise top, high recorded bottom currents). If sufficient evidence exists to suspect overconsolidated soils, it would be prudent to drop some penetrometers or short gravity corers. However, the engineer should be aware that both of these devices will not penetrate deeply into highly overconsolidated sediment. Nonpenetration or slight penetration with attainment of minimal sample length can add credence to the suspicion that overconsolidated sediment does indeed exist. Typical sand properties are given in Figure 2-5. If the location is near a large active river delta, the site must be surveyed directly.

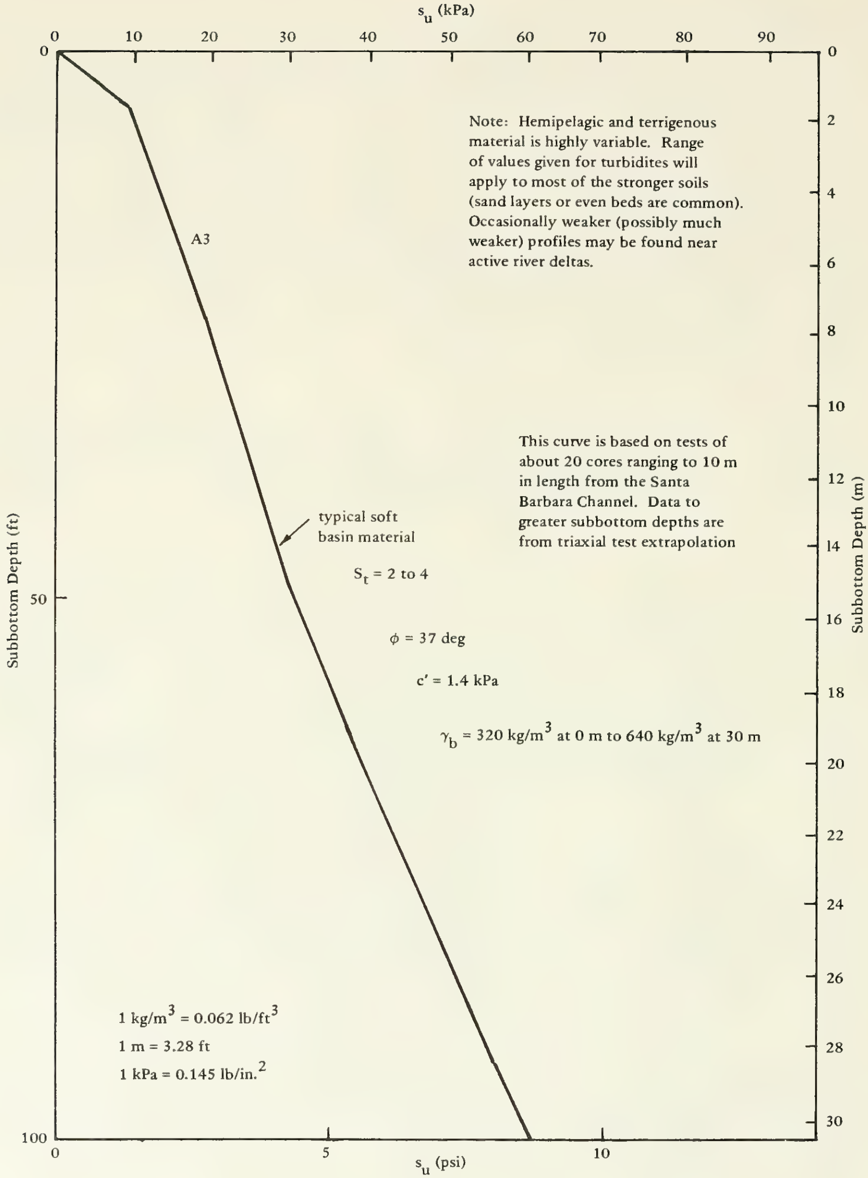

Figure 2-4. Typical strength profile for hemipelagic, terrigenous silty clay (from Lee and Clausner, 1979).

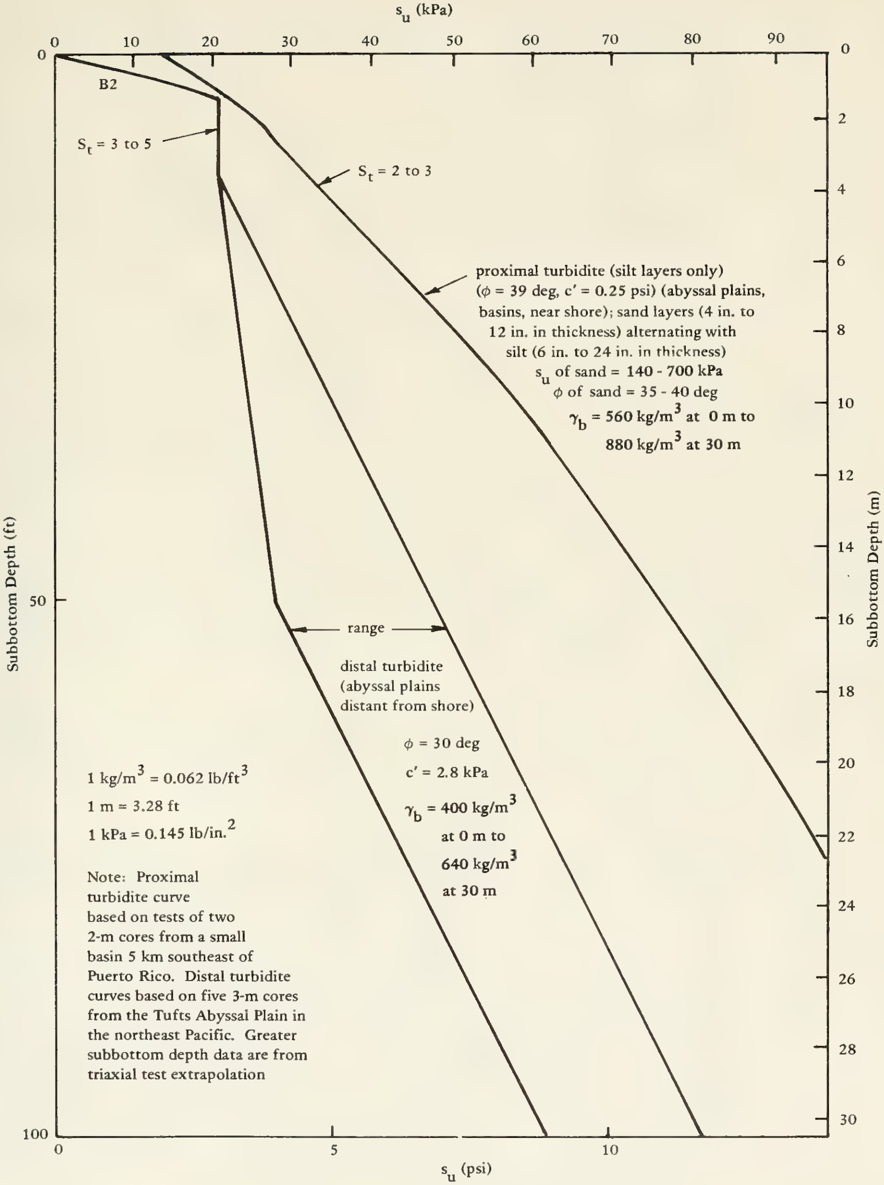

Figure 2-5. Typical strength profiles for turbidites (from Lee and Clausner, 1979).

(2) If the site is in the deep ocean and not on an abyssal plain, it must be determined whether its water depth lies above or below the CCD (Figure 2-2).

(a) If above the CCD, the sediment is probably calcareous ooze. Figure 2-6 gives the typical properties; it should be noted that a further subdivision between coarse and fine ooze is made at the 3,000-meter (10,000-foot) level.

(b) If the site is below the CCD, the sediment is probably pelagic clay. Figure 2-7 shows the typical properties.

(3) If the location is identified on physiographic province charts as an abyssal plain, the typical properties (classed as turbidite) shown in Figure 2-5 are assumed. A split is made between proximal and distal turbidites. The distance from a source of sand (the shore or perhaps the edge of the continental shelf) distinguishes the two: if the distance is greater than about 50 km (30 miles), the sediment is probably a distal turbidite.

(4) If the location is classed as a siliceous ooze (diatom or radiolarian ooze, Figure 2-1), the typical properties can be found in Figure 2-8.

(5) Whenever possible, recognized experts should be consulted as they can provide information that is difficult to glean from the open literature. Many parts of the seafloor have been mapped for sediment distributions, and much more detailed information than can be given in this discussion may be available. In addition, many core sample descriptions are available. Sources for experts, maps, and core descriptions include:

Lamont-Doherty Geological Observatory of
Columbia University
Palisades, NY 10964

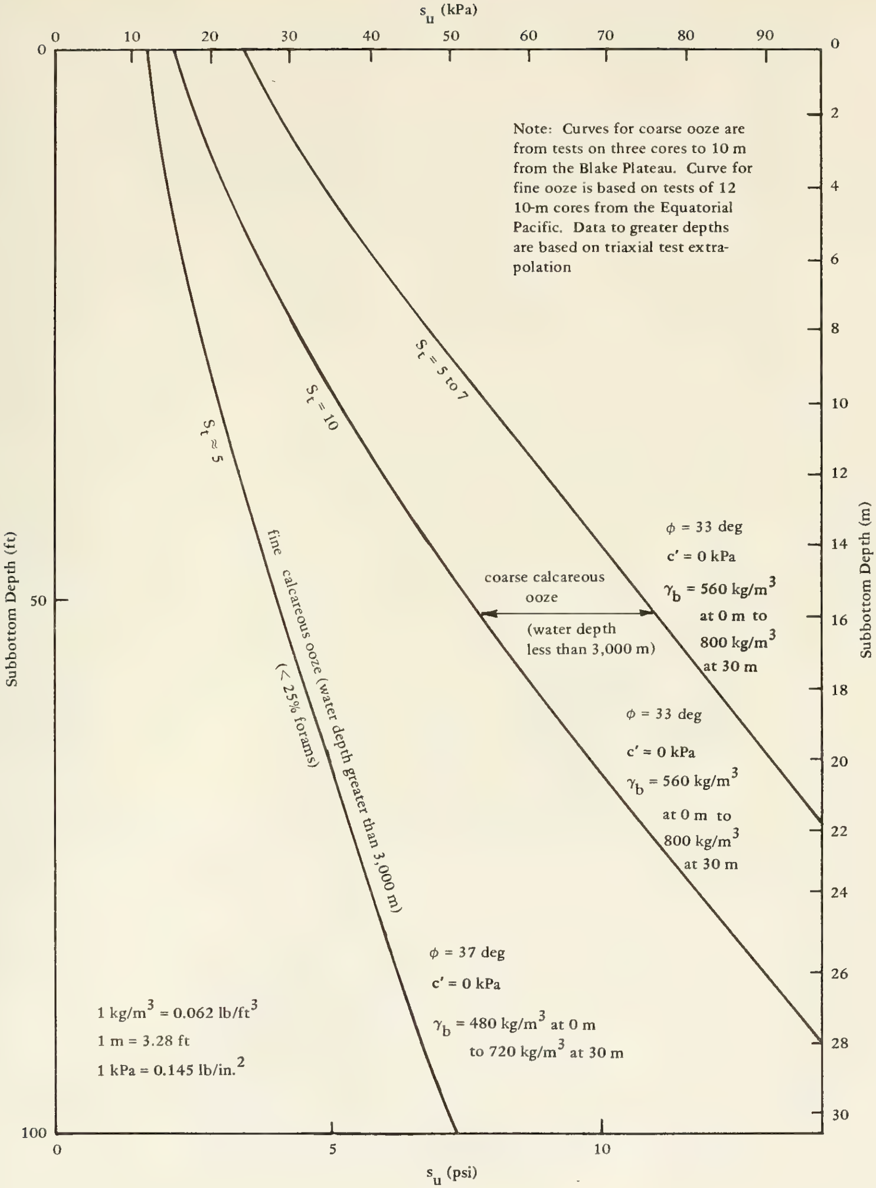

Figure 2-6. Tyipcal strength profiles for calcareous ooze (from Lee and Clausner, 1979).

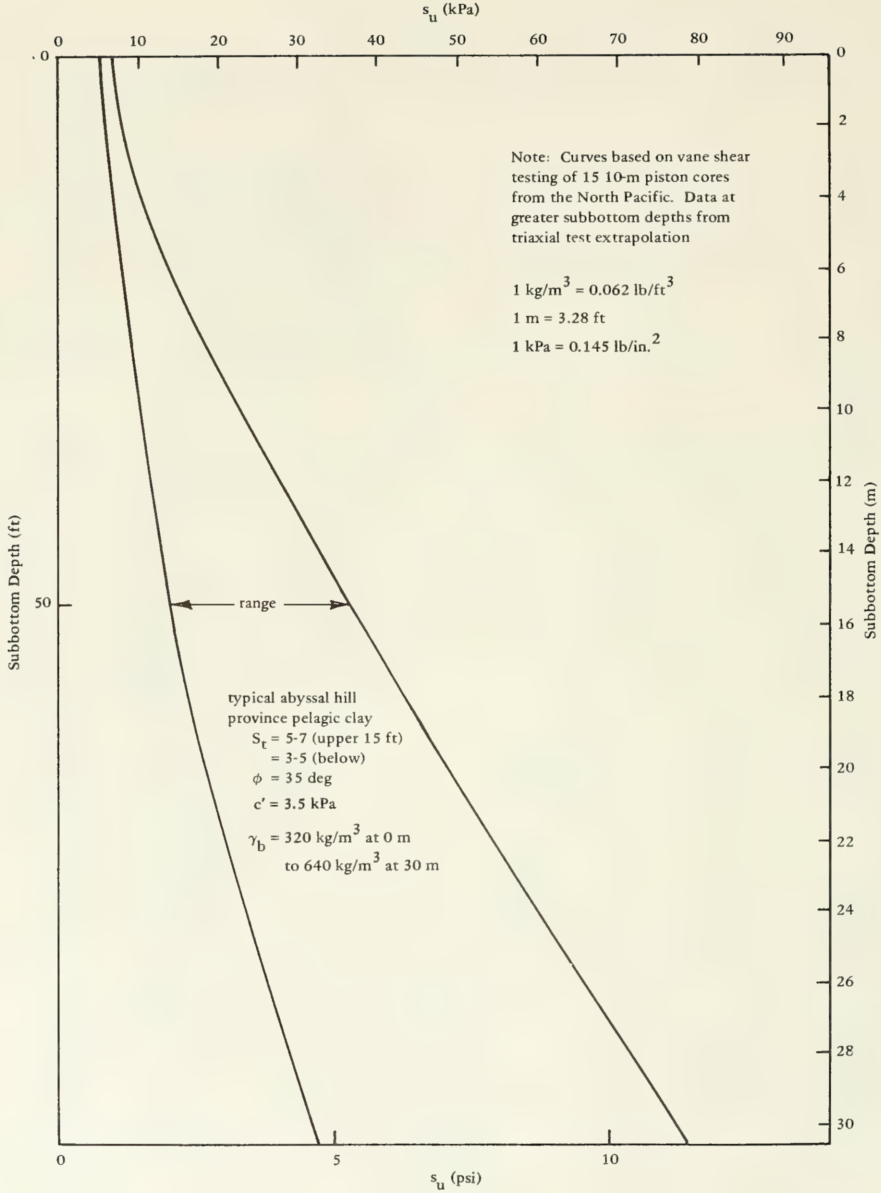

Figure 2-7. Typical strength profiles for pelagic clay (from Lee and Clausner, 1979).

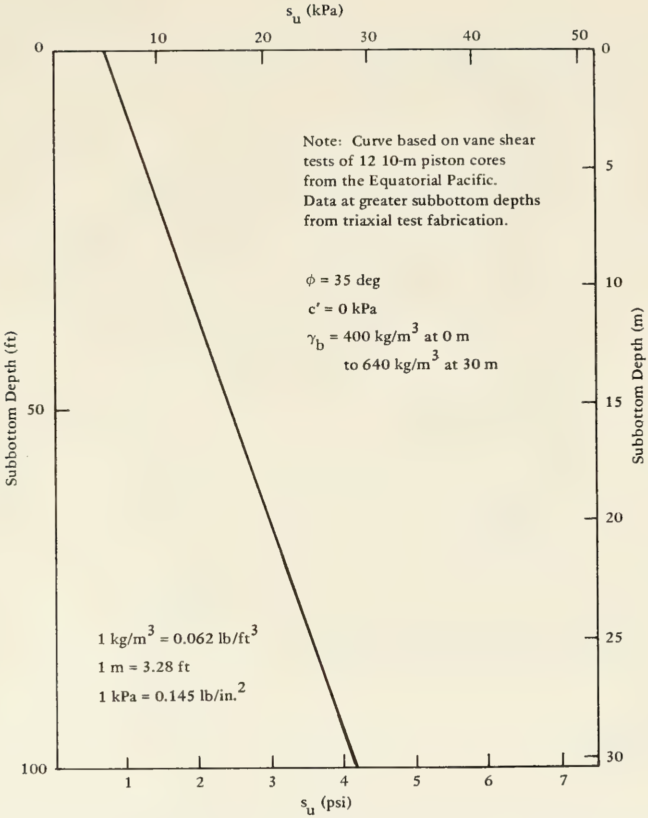

Figure 2-8. Typical strength profile for siliceous ooze (from Lee and Clausner, 1979).

National Geophysical and Solar-Terrestrial
Data Center
Environmental Data Service
National Oceanic and Atmospheric
Administration
Boulder, CO 80302

Chief of Operations Division
National Ocean Survey, NOAA
1801 Fairview Avenue, East
Seattle, WA 98102

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345 Middle Road
Menlo Park, CA 94025

Woods Hole Oceanographic Institution
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2.4 HAZARDOUS/UNUSUAL CONDITIONS

The procedures presented herein assume the use of ordinary conditions to achieve a degree of simplicity. When conditions are out of the range for the assumed conditions, caution is necessary. These cases are termed hazardous/unusual. Satisfactory designs are still possible, but they require reference to the more detailed procedures given in the references listed in Section 1.3. A hazardous/unusual condition exists when any of the following are true.

(a) Bedrock or rock pieces larger than gravel size present at depths less than fluke penetration.

(b) Soil type changes significantly or there are major layers of different soil types.

(c) Deep ocean siliceous ooze (>30% biogenic and siliceous).

(d) Clean calcareous ooze (>60% biogenic and calcareous).

(e) Sensitivity >6 in a cohesive soil.

(f) Cohesive soil strength profile that varies by more than -50% or +100% from the typical profiles presented.

(g) Site slopes greater than 10 degrees.

3. PENETRATION

Penetration of propellant-embedded anchor flukes is a complex phenomena that has received much research attention. True (1976) has developed an analytical model for penetration prediction for high velocity penetrators based on a form of Newton's second law

$$M' \, v \frac{dv}{dz} = W_b + F_1 + F_2 + F_3 \tag{3-1}$$

where M' = penetrator effective mass

v = penetrator velocity

d = differential operator

z = depth

W_b = buoyant weight of penetrator

F_1 = inertial drag force

F_2 = bearing component force

F_3 = side adhesion force

3.1 COHESIVE SOILS

True developed a solution to Equation 3-1 for cohesive soils by conducting a number of model and field penetration tests. The solution is not closed form; thus, an incremental technique is necessary to solve it. A rigorous solution would consider different values of many of the parameters over the length of the penetrator and is suited for a computer. Such a solution is beyond the scope of this report. However, because anchor flukes usually penetrate a minimum of five times their length, many simplifications are possible by assuming the penetrator is a point object at the i^{th} depth increment. With a two-sided finite difference form and substitution of parameters for M', F_1, F_2, F_3, and W_b per True (1975, 1976, and 1977), Equation 3-1 becomes

$$v_{i+1} = v_{i-1} + \frac{2\,\Delta z}{v_i (M + 2\,\rho_i\,V)} \left\{ \left[W - V\,\gamma_i\right] - \left[\frac{1}{2}\,v_i^{\,2}\,A_f\,C_D\,\rho_i\right] \right.$$

$$\left. - s_{ui}\left[A_f\,N_c + \frac{A_s\,\delta^*}{S_{ti}}\right]\left[\frac{S_e^{\,*}}{1 + \dfrac{1}{\sqrt{\dfrac{C_e^{\bullet}\,v_i}{s_{ui}\,t} + 0.06}}}\right] \right\} \tag{3-2}$$

where

i	=	subscript to indicate the i^{th} increment of depth
M	=	penetrator mass
ρ_i	=	mass density of penetrated medium
V	=	volume of penetrator
W	=	penetrator weight
γ_i	=	unit weight of penetrated medium
A_f	=	penetrator frontal area
C_D	=	inertial drag coefficient
s_{ui}	=	soil undrained shear strength
N_c	=	bearing capacity factor
A_s	=	side area of penetrator
δ^*	=	side adhesion factor
S_{ti}	=	soil sensitivity
t	=	penetrator thickness
$S_e^{\,*}$	=	empirical maximum soil strain rate factor
$C_e^{\bullet}$	=	empirical soil strain rate factor

Sufficient accuracy is obtained when about 10 increments of depth are used to arrive at a velocity less than zero. To begin the iterative procedure set $i = 1$ and $v_1 = v_0$ and solve Equation 3-2 for v_2. Then re-evaluate v_1 as

$$v_1 = \frac{v_0 + v_2}{2} \tag{3-3}$$

and solve Equation 3-2 again for $i = 1$ using this new value for v_1. Continue to iterate for $i = 2, 3, \ldots$ until $v_{i+1} \leqq 0$. The depth of embedment, D_p, can then be determined from

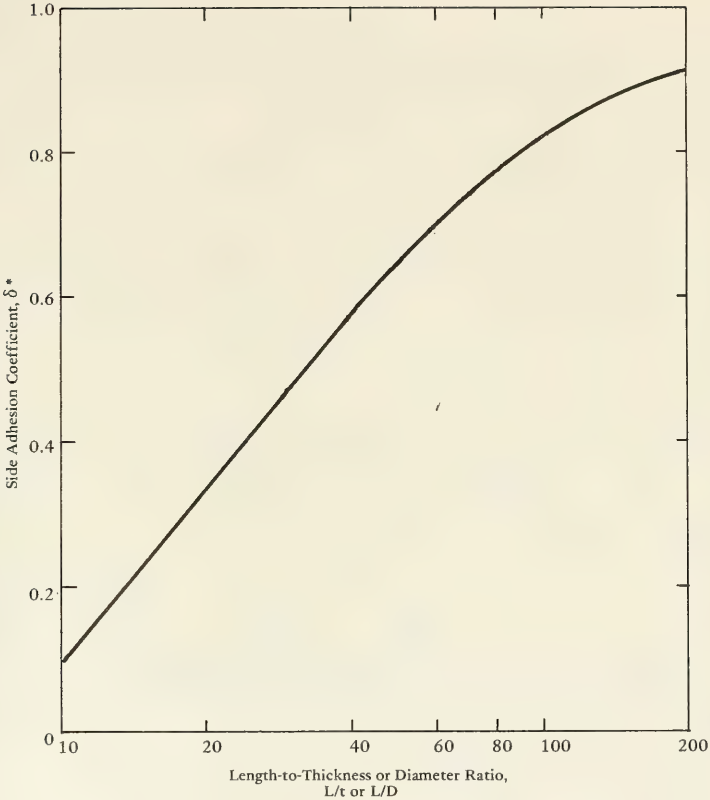


Figure 3-1. Side adhesion coefficient as a function of length-to-thickness or diameter ratio.

$$D_p = i(\Delta z) \tag{3-4}$$

True (1977) recommends a value of 0.7 for C_D and that S_e* be taken as 4, and that $C_{\dot{e}}$ be 20 N-sec/m² (4 lb-sec/ft²). Values for δ* are given in Figure 3-1 as a plot of δ* versus length-to-thickness or diameter ratio. The values for δ* can also be calculated from the formula

$$\delta^* = 1 - \frac{1 + 3.5\left[1 - e^{-\frac{1}{3.5}\left(\frac{L}{4\ t\ \tan\alpha} - 1\right)}\right]}{\frac{L}{4\ t\ \tan\alpha}} \qquad (3\text{-}5)$$

where L = penetrator length

α = 1/2 angle of the penetrator tip

A tabulation of penetration depth is provided in Table 3-1 for each CEL propellant-embedded anchor for each typical soil profile given in Section 2. The calculations assumed a clay fluke and the gun operated at peak efficiency.

Table 3-1. Penetration Calculations for the Clay Flukes of Each CEL Propellant-Embedded Anchor at Peak Performance for the Typical Profiles Given in Section 2

Soil Type	Anchor Penetration, m (ft) for --			
	300K	100K	20K	10K
Soft basin soil	19.5 (64)	15.9 (52)	10.7 (35)	7.6 (25)
Distal turbidite (low)	17.4 (57)	13.1 (43)	8.2 (27)	5.8 (19)
Distal turbidite (high)	14.9 (49)	11.9 (39)	7.9 (26)	5.8 (19)
Proximal turbidite	12.5 (41)	10.1 (33)	7.0 (23)	5.2 (17)
Calcareous ooze (deep water)	22 (72)	18.3 (60)	11.9 (39)	8.2 (27)
Course calcareous ooze (low)	19.2 (63)	16.5 (54)	10.7 (35)	7.6 (25)
Course calcareous ooze (high)	15.2 (50)	12.8 (42)	8.2 (27)	5.8 (19)
Siliceous ooze	24.1 (79)	19.8 (65)	13.1 (43)	9.2 (30)
Pelagic clay (low)	24.7 (81)	20.7 (68)	14.3 (47)	10.1 (33)
Pelagic clay (high)	19.2 (63)	15.9 (52)	11.3 (37)	8.2 (27)

3.2 COHESIONLESS SOILS

Procedures for predicting fluke penetration in cohesionless soils are not well advanced. Predictions have been made using modified versions of True's method (involving the undrained shear strength of the soil), but the procedures are undocumented and unverified. In view of this lack of an analytical method, penetrations must be estimated from previous data. Fortunately, the performance of the CEL propellant-embedded anchors in sand is very good, and conservative estimates of penetration will yield holding capacity estimates well above the nominal capacities of the anchors and yet conservative with respect to the available anchor performance data. Penetration estimates based on the experience of the Civil Engineering Laboratory are provided in Table 3-2. An assumption is made that full design charges are used in the anchors. These estimates are recommended for use in predicting anchor performance in the absence of actual installation experience at any given site.

Table 3-2. Propellant-Embedded Anchor Penetration Depth (D_p) Estimates (to Fluke Tip) in Cohesionless Soils

Anchor	Anchor Penetration, m (ft) in --		
	Loose Sand[a]	Medium Dense Sand[b]	Dense Sand[c]
CEL 10K sand/coral fluke	3.8 (12.5)	3.4 (11)	3.1 (10)
CEL 20K sand/coral fluke	5.2 (17)	4.9 (16)	4.6 (15)
CEL 100K sand/coral fluke	7.6 (25)	7.0 (23)	6.4 (21)
CEL 300K universal fluke	9.2 (30)	8.2 (27)	7.6 (25)

[a] ϕ = 30 degrees; γ_t = 1,760 kg/m^3 (110 lb/ft^3)
[b] ϕ = 35 degrees; γ_t = 1,920 kg/m^3 (120 lb/ft^3)
[c] ϕ = 40 degrees; γ_t = 2,080 kg/m^3 (130 lb/ft^3)

4. FLUKE KEYING

Fluke keying is the process of turning the fluke from the edge-on orientation of penetration to an orientation that presents a large area to resist pullout. During keying, fluke embedment depth is reduced. Therefore, holding capacity is calculated at this reduced depth, the embedment depth of the fluke. The embedment depth is arrived at by subtracting the fluke keying distance from the penetration depth.

It is generally thought that the breakout load must be applied to achieve full keying. However, it is not necessary to fully key the anchor/fluke for it to perform properly. In-service loads that exceed the keying load applied at installation will achieve additional keying.

In cohesive soils there may be an influence of time on keying distance. Rocker (1977) showed that in a clay with a sensitivity of 2 to 3, waiting about 1 hour after embedment improved the keying distance to two fluke lengths compared to three to four fluke lengths after waiting only 1/4 hour. Since Rocker's study, improvements have been made in the fluke design to shorten keying distance. With the improved flukes, CEL field tests in a soft clay of similar sensitivity showed the flukes keyed in two fluke lengths or less with less than 1/4 hour wait after embedment (Clausner, 1978). In view of this recent data, fluke keying distance in cohesive soils can be taken as two fluke lengths.

For cohesive soil, the embedment depth is estimated from:

$$D = D_p - 2\ L \qquad (4\text{-}1)$$

For cohesionless soils, the embedment depth is estimated from:

$$D = D_p - 1.5\ L \qquad (4\text{-}2)$$

5. STATIC HOLDING CAPACITY

5.1 SHORT-TERM STATIC LOADING

Short-term static holding capacity is defined as the load required to cause anchor breakout when the anchor is loaded rapidly to failure. For cohesive soil, the rapid loading does not allow drainage, and the capacity is governed by a soil's undrained shear strength. In cohesionless soil, drainage occurs even though the soil is loaded rapidly, and the capacity is governed by a soil's friction angle.

For short-term static holding capacity use a factor of safety between 2 and 3, depending on the nature of the anchorage and the reliability with which the soil parameters have been determined.

5.1.1 Cohesive Soil

For short-term holding capacity in cohesive soils Equation 1-1 reduces to:

$$F_{st} = A \bar{N}_c s_u f(0.84 + 0.16 B/L) \qquad (5\text{-}1)$$

where F_{st} = short-term static holding capacity

A = projected fluke area

f = correction factor to account for soil disturbance

s_u = soil undrained shear strength

$\bar{N}_c$ = short-term holding capacity factor in cohesive soil

B = fluke width or diameter

L = fluke length or diameter

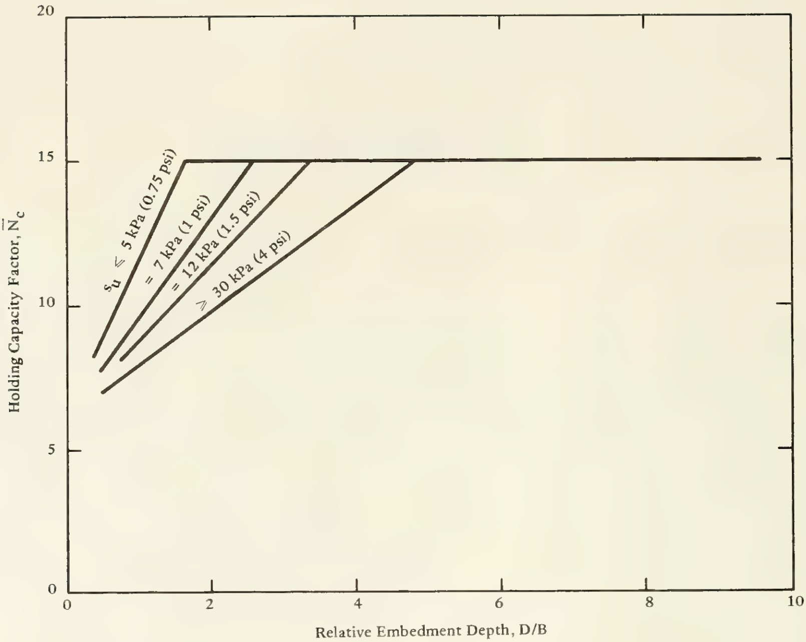


Figure 5-1. Short-term holding capacity factors for cohesive soils (from Beard, 1979).

The soil cohesion, c, of Equation 1-1 becomes the undrained shear strength, s_u, for this type of loading. The values of $\bar{N}_c$ are given in Figure 5-1 as a function of relative fluke embedment depth, D/B. The values of f have been recommended by Valent (1978) as 0.8 for terrigenous clayey-silts and silty-clays, 0.7 for pelagic clays, and 0.25 for calcareous ooze. This factor accounts for soil disturbance from fluke penetration and keying. The validity of f = 0.25 for calcareous ooze is in doubt, as it suggests a factor of 4 loss of soil strength. However, until data supporting a higher value are available, its use is recommended. A, B, and L are anchor fluke parameters.

5.1.2 Cohesionless Soil

For cohesionless soil ($c = 0$), the short-term static holding capacity can be calculated using Equation 1-1 in this form:

$$F_{st} = A\, \gamma_b\, D\, \bar{N}_q (0.84 + 0.16\, B/L) \qquad (5\text{-}2)$$

where
F_{st} = short-term static holding capacity
A = projected fluke area
γ_b = buoyant unit soil weight
D = depth of fluke embedment
$\bar{N}_q$ = holding capacity factors for drained or frictional condition
B = fluke width or diameter
L = fluke length or diameter

The values to use for $\bar{N}_q$ are given as a function of relative embedment depth, D/B, and soil friction angle, ϕ, in Figure 5-2. A, B, and L are anchor fluke parameters. D is the depth from the soil surface to the anchor fluke. The soil parameters should be selected in accordance with the procedures provided in Chapter 2.

5.2 LONG-TERM STATIC LOADING

Long-term static holding capacity is the largest load an anchor fluke can sustain for a long period of time. Loading a fluke stresses the soil and, thereby, generates excess porewater pressure. The long-term condition is reached when the excess porewater pressure has dissipated. The time for this to occur in a particular soil is a function of the soil's permeability and other characteristics. In cohesionless materials, this drainage occurs almost immediately, and no distinction is made between long-term and short-term holding capacity. In cohesive

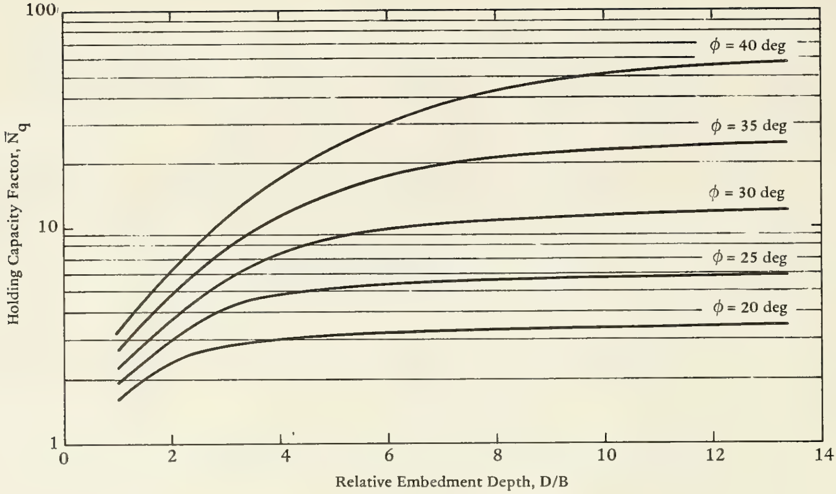


Figure 5-2. Holding capacity factors for cohesionless soils (from Beard, 1979).

soils, an extended period of time is required to obtain a similar condition. When the condition is reached, drainage is complete, and anchor capacity is governed by a cohesive soil's drained strength parameters. Those parameters are the friction angle, ϕ, and the cohesion intercept, c'.

For long-term static holding capacity, use a factor of safety of between 2 and 3, depending on the nature of the anchorage and the reliability with which the soil parameters have been determined.

5.2.1 Cohesive Soil

For cohesive soils, Equation 1-1 is used directly.

$$F_{\ell t} = A(c' \bar{N}'_c + \gamma_b D \bar{N}_q)(0.84 + 0.16 B/L) \tag{5-3}$$

where $F_{\ell t}$ = long-term static holding capacity

A = projected fluke area

B = fluke width or diameter

L = fluke length or diameter

c' = soil cohesion intercept

γ_b = soil buoyant unit weight

D = fluke depth below soil surface

$\bar{N}'_c$ = long-term holding capacity factor in cohesive soil

$\bar{N}_q$ = holding capacity factor for drained or frictional condition

The prediction of long-term holding capacity is based on the principle that the behavior of cohesive and cohesionless soils is basically the same. Hence, in cohesive soils with full drainage, the effective stress principle can be applied using the drained strength parameters ϕ and c'. The $\bar{N}'_c$'s (Figure 5-3) to be applied to the cohesion are different than the $\bar{N}_c$'s applied to the undrained shear strength when predicting short-term holding capacity. For short-term loading, the holding capacity factors include the effect of suction below the anchor fluke. The factors to be used here do not include it, because, for long-term use, suction below the anchor is dissipated. The values of $\bar{N}_q$ are presented in Figure 5-2 and are the same as those for analyzing short-term loading in cohesionless soils.

When the soil is loose (soft), the shear failure of the soil is different than when the soil is firm. To account for this difference, drained strength parameters, ϕ and c', should be reduced by one-third before selecting holding capacity factors as is done in bearing capacity analyses. For ϕ, the reduction is applied to the tangent of ϕ, and then a revised ϕ is obtained by determining the arc tangent of the result.

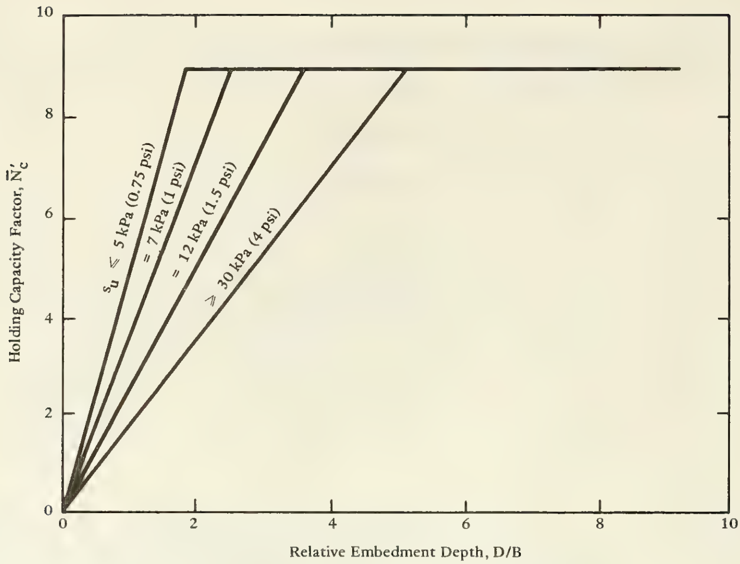


Figure 5-3. Holding capacity factors for cohesive soils under long-term loading (from Beard, 1979).

5.2.2 Cohesionless Soils

No distinction is made between short-term and long-term loading in cohesionless soils. Therefore, to estimate long-term capacity in cohesionless soils, use Equation 5-2.

6. DYNAMIC HOLDING CAPACITY

6.1 IMPULSE LOADING

Impulse loads for cohesive soils are defined as loads greater than the short-term capacity having a duration of less than 10 minutes. Impulse loads for cohesionless soils are defined as loads greater than the short-term capacity having a duration of less than 10 seconds. The same basic procedure used for determining short-term static holding capacity is followed for estimating anchor holding capacity under impulsive loading. The procedures that follow are appropriate for use with circular, square, or near square ($\ell/d = 2$) anchor flukes only. Most flukes in use meet this criterion. The procedures, developed by Douglas (1978), consist of applying various influence factors to the basic equations. These factors have been chosen to yield conservative results, even for the worst conditions. No additional factors of safety are required beyond those recommended for short-term static holding capacity.

6.1.1 Cohesive Soil

To determine the impulse holding capacity of an anchor embedded in cohesive soil, the short-term static holding capacity must be determined first. The short-term static capacity is adjusted by the equation:

$$F_I = F_{st} \; I \; R_c \; R_I \; I_f \tag{6-1}$$

where F_I = impulse loading holding capacity

F_{st} = short-term static holding capacity

I = influence factor for adjusting the soil strength

R_c = reduction factor for determination of cyclic loading holding capacity

R_I = reduction factor for repeated impulses

I_f = inertial factor for capacity increase under very rapid loading

I is determined from Figure 6-1. To use this figure the load duration of the pulse of concern must be estimated and the general soil characteristics be known or estimated. It is not possible to generalize seafloor soil characteristics, but when the undrained shear strength has been estimated, the curve for normally consolidated, moderately sensitive clays is used. If the impulse load is the first event in an anchor's history or when cyclic loading is not expected, $R_c = 1$. However, when cyclic loads are expected prior to the impulse event, R_c is the inverse of the allowable double amplitude cyclic load expressed as a fraction of the static short-term capacity as determined in paragraph 6.2.1. R_I, the reduction factor for repeated impulses, can be calculated from:

$$R_I = 1.33\, e^{-1.15\,\bar{f}} \tag{6-2}$$

where $\bar{f}$ = average frequency, in impulses per hour, for a 4-hour period

The inertial factor, I_f, should only be applied if the impulse duration is less than 0.01 second. Values of I_f can be found directly from Figure 6-2.

6.1.2 Cohesionless Soils

The impulse loading capacity in cohesionless soil is determined by the application of factors to the short-term holding capacity.

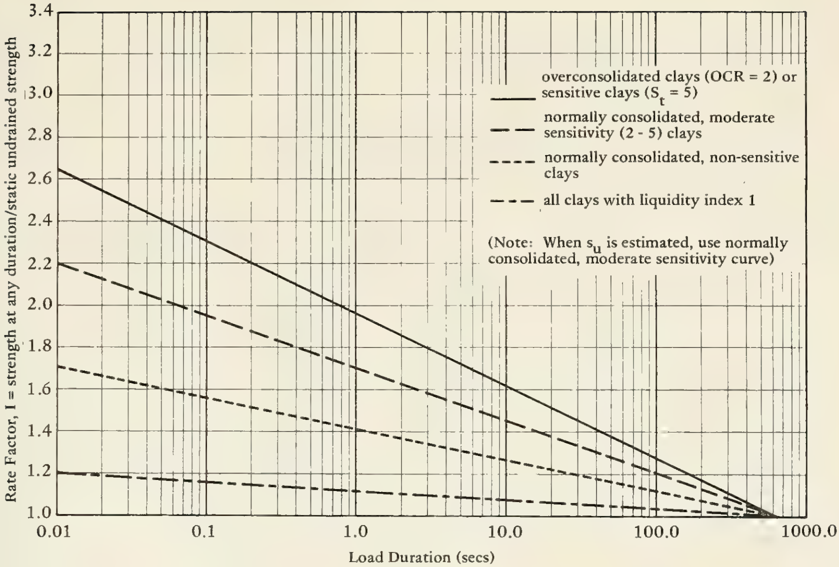


Figure 6-1. Rate factors for cohesive soils (from Douglas, 1978).

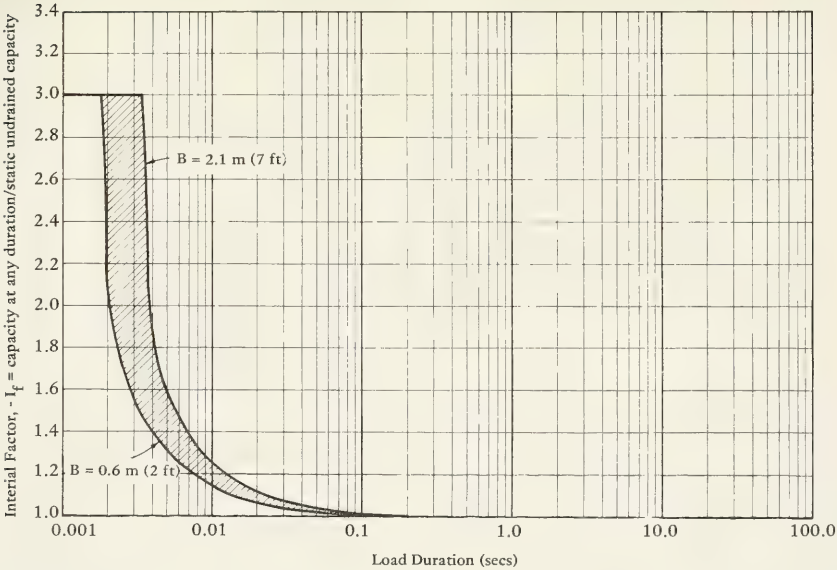


Figure 6-2. Inertial factors for sand and clay (from Douglas, 1978).

$$F_I = F_{st}\left(\frac{\bar{N}_{qI}}{\bar{N}_q}\right) R_c\, R_I\, I_f \qquad (6\text{-}3)$$

where F_I = impulse loading holding capacity

F_{st} = short-term holding capacity

$\bar{N}_{qI}$ = cohesionless soil holding capacity factor adjusted for impulse loading

$\bar{N}_q$ = cohesionless soil holding capacity factor

R_c = reduction factors for determination of cyclic loading holding capacity

R_I = reduction factor for repeated impulse loads

I_f = inertial factor for capacity increase under very rapid loading

$\bar{N}_q$ is the same as that used to determine the short-term holding capacity (Figure 5-2). $\bar{N}_{qI}$ is an adjusted $\bar{N}_q$ based on the impulse loading effect on the friction angle, ϕ. Values for $\bar{N}_{qI}$ can be obtained from Figure 5-2 by using the adjusted friction angle, ϕ_I, in place of ϕ. ϕ_I is calculated from:

$$\phi_I = \sin^{-1}\left(\frac{I \sin \phi}{1 + (I-1) \sin \phi}\right) \qquad (6\text{-}4)$$

where ϕ_I = soil impulse adjusted internal angle of friction

ϕ = soil internal angle of friction

I = influence factor for adjusting soil strength

Values of I are given in Figure 6-3. R_c, the cyclic loading reduction factor, is taken as 1 when the impulse load is the first event in an anchor history or when cyclic loading is not expected. Otherwise R_c is the inverse of the allowable double amplitude cyclic load expressed as a fraction of the short-term holding capacity as determined in paragraph 6.2.1. If an impulse is not repeated within 10 minutes, $R_I = 1$. When the loading is expected to be repeated within 10 minutes

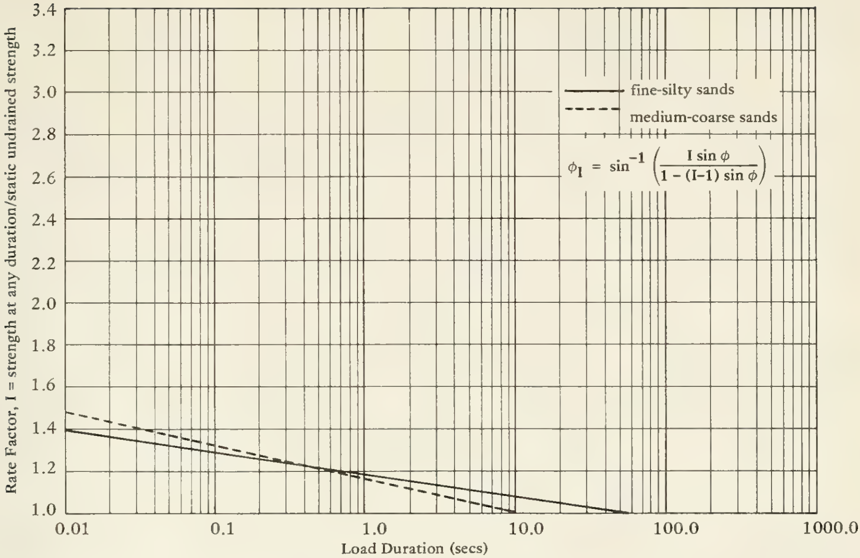


Figure 6-3. Strength influence factors for cohesionless soils (from Douglas, 1978).

$$R_I = 2\, e^{-0.116\, \bar{f}} \qquad (6\text{-}5)$$

where $\bar{f}$ = number of impulses in 10 minutes

I_f is used only when the impulse load duration is expected to be 0.01 second or less. When this condition is met, values for the inertial factor can be found in Figure 6-2.

6.2 CYCLIC LOADING

Cyclic loads are loads of less than 1-minute duration applied in a repetitive manner. They result primarily from wave-induced forces, cable strumming, and earthquake loading. Waves induce cyclic loads by

their action on surface or subsurface moored objects. The frequencies of these loads are usually at the frequency of the waves; typically 0.05 to 0.15 Hertz. Cable strumming is caused by currents passing a relatively taut cable. The cyclic loads from cable strumming are low magnitude and can be ignored when using the procedures of this report. Earthquake loading differs from the previous two cases in that the entire soil mass is loaded rather than just the anchor fluke and adjacent soil. This special type of cyclic loading is covered in Section 6.3.

Cyclic loads must have a double amplitude greater than 5% of the quasi-static anchor capacity to be of concern from a cyclic capacity design standpoint. Cyclic loads below this threshold can be ignored.

Cyclic loads are characterized by a pure cyclic loading component superimposed on a quasi-static loading component. Cyclic and static magnitudes are expressed as percentages of the static anchor holding capacity. Figure 6-4 gives an example of cyclic loading where the quasi-static load component is 20% of the static capacity and the double amplitude cyclic load component is 40% of the static capacity.

Two additional characterizations concern the number of load cycles. One, the total number of load cycles in the anchor's lifetime, n_T, is used in evaluating cyclic creep potential. The other, the number of cycles, n_C, that occur in a period, t_{cd}, marked by low excess pore-pressure dissipation is used to evaluate strength loss or liquefaction failure potential. Cyclically loaded anchors are designed to preclude failures from creep or liquefaction. Creep failure is an accumulation of small movements that reduce anchor depth and, hence, capacity until pullout occurs. A liquefaction-like failure is characterized by soil strength loss and sudden anchor instability.

The above guidance is straightforward when the cyclic loads are of relatively uniform magnitude or when a major portion of the cyclic load (one-third) is relatively uniform and significantly larger (50%) than the rest. For cases where distribution of cyclic loads is not uniform and exhibits extreme values, a different approach is required. In general, for these cases, the loading spectrum can be broken into segments of loads of relative uniformity. These segments can be characterized by

their quasi-static load, the superimposed double amplitude cyclic load, and the cumulative number of load cycles at or greater than the given cyclic load.

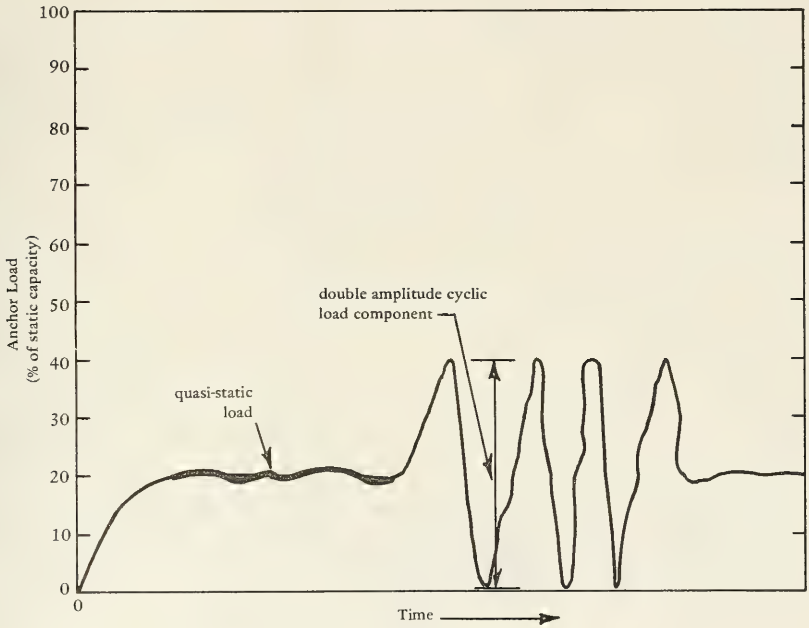


Figure 6-4. Examples of cyclic loading.

The design procedures that follow were developed by Herrmann (1980). The procedures incorporate a number of conservative assumptions and "worst case" values in obtaining the limiting conditions. Because of this, lower factors of safety are allowable. Whereas values of 2 to 3 were recommended for static cases, for similar conditions, values of 1.25 to 1.75 are recommended for cyclic load conditions. These factors are to be applied to the loads, not to the number of cycles.

6.2.1 Strength Loss During Anchor Cyclic Loading

Some soils, such as uniform fine sand, coarse silts, and some clean oozes, are susceptible to true liquefaction failure when subjected to cyclic loads. These soils are excluded from the prediction procedures that follow. Recommendations for these soils are given by Herrmann (1980) (to be published). Most other soils, including very plastic cohesive soils, are subject to strength loss, especially from extended cyclic loading. In general, denser soil, more plastic soil, and lower cyclic loads lead toward lower susceptibility to strength loss.

For all soils, the loading is characterized using the above definitions and Figure 6-5 to determine t_{cd} from the soil's permeability. The number of load cycles during a period equal to t_{cd} is found, and limiting design bounds as a function of soil type are then established using Figure 6-6. Figure 6-6 can be used to find the limiting number of cycles for a given loading or the limiting loading for a given number of cycles. The upper bounds apply to cases where the average quasi-static load is 1/3 or less of the static holding capacity. For the unlikely case where the average quasi-static load is greater than 1/3 of the static holding capacity, the excess is added to the double amplitude cyclic load prior to using Figure 6-6.

6.2.2 Cyclic Creep During Anchor Cyclic Loading

The mechanism leading to cyclic creep of an embedment anchor is not well understood but is known to occur under loading conditions that in some cases are quite safe relative to the criteria for cyclic strength loss presented in the preceding section. For cyclic creep considerations, the number and magnitude of significant loading cycles occurring during the lifetime of an anchor are controlling factors. They should be summarized in spectral or quasi-spectral format. The number of significant loading cycles may not be as large as one would expect. If, for example, a mooring system has a planned 20-year life, is continuously in use, and is subjected to strong wave loading during ten 3-day storms per year, then the total number of significant cyclic loads will likely be less than 1 million.

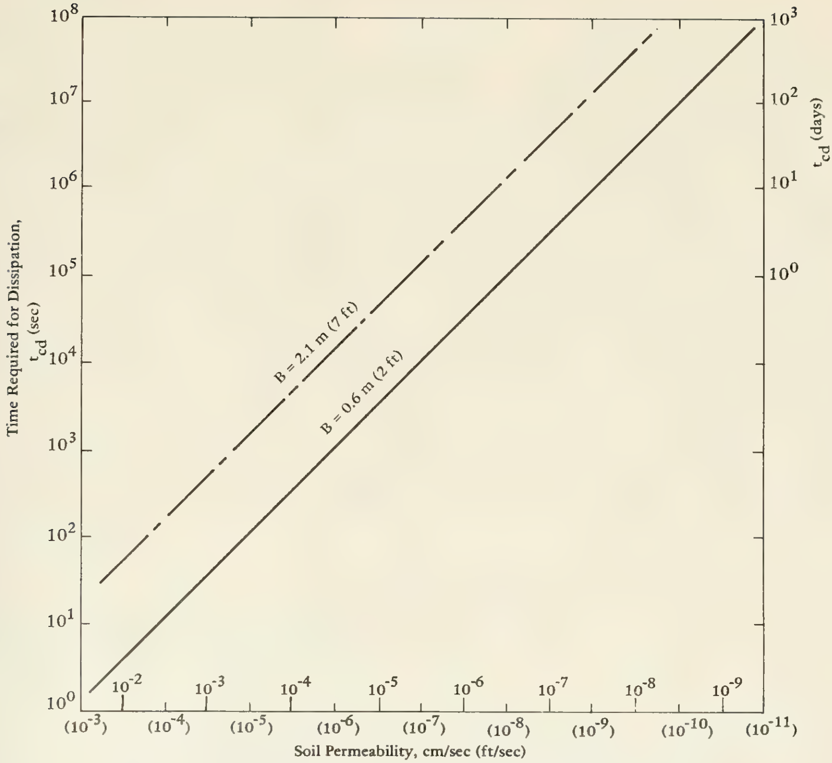


Figure 6-5. Time required for stress porepressure redistribution/dissipation (from Herrmann, unpublished).

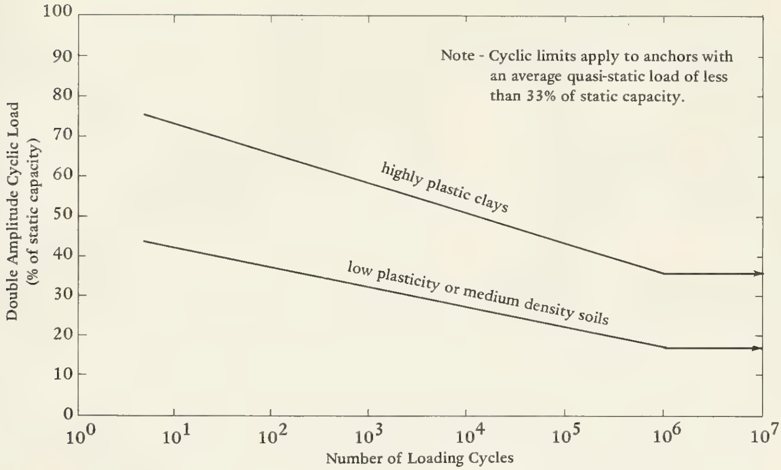


Figure 6-6. Contours of cyclic capacity without porepressure dissipation (from Herrmann, unpublished).

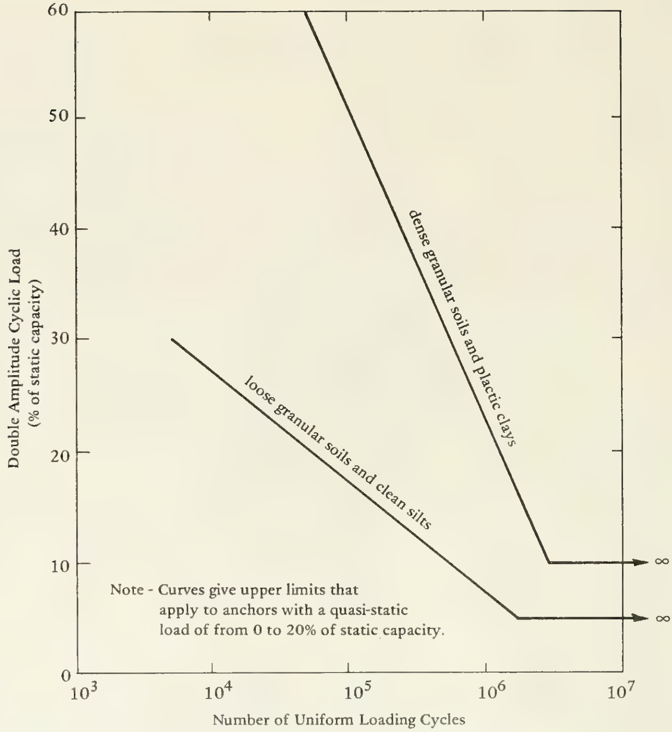


Figure 6-7. Contours of cyclic loading without cyclic creep limits or number and magnitude of loading cycles to prevent failure due to cyclic creep (from Herrmann, unpublished).

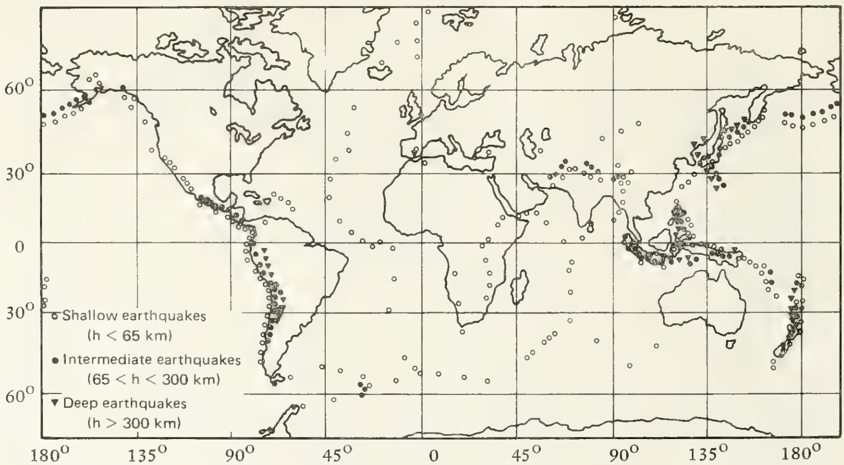


Figure 6-8. Geographic distribution of earthquake epicenters (from Wilson, 1969).

The established criteria for maximum cyclic loading are presented in spectral format in Figure 6-7. Criteria are presented for two categories of soil type. The more restrictive criterion applies to few sites for which these guidelines are applicable, as most such sites are excluded as hazardous sites as defined in an earlier chapter. The criteria established in Figure 6-7 are applicable to cases where the average quasi-static load is less than 20% of static anchor capacity. For cases when this static load is exceeded, the portion above 20% should be singly added to the double amplitude cyclic load and the analysis continued. This requirement is quite restrictive for longer life anchor systems subjected to significant and long-term cyclic loading; however, cyclic creep of anchors is not well understood and until further data are available, this relatively conservative approach is recommended.

6.3 EARTHQUAKE LOADING

Earthquakes cause loads (usually at a frequency of about 2 Hertz and with 10 to 30 significant loading cycles depending upon the magnitude of the earthquake) that differ from the preceding category in that the cyclic loading is induced relatively uniformly in the entire soil mass by the earthquake energy emanating from the epicenter. The geographical locations of past major earthquakes, and thus likely future ones, are illustrated in Figure 6-8. The maximum accelerations induced in the soil mass by major earthquakes are a function of the earthquake magnitude and the distance of the site from the earthquake epicenter, or more precisely from the causative fault zone. Predictions of these accelerations are summarized in Figure 6-9.

A major earthquake centered within 60 km (100 miles) of an anchor can temporarily reduce the anchor capacity relative to all types of loads. This possibility exists only for relatively clean, granular soils (granular soils with few fines - clay size or fine silts). Cohesive soils do not lose any significant amount of strength (relative to anchor capacity) in the 30 or less significant loading cycles associated with

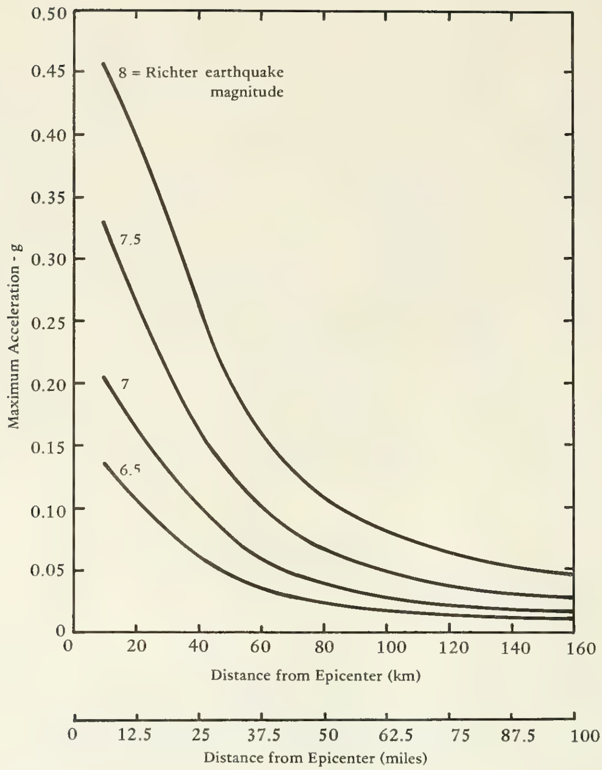
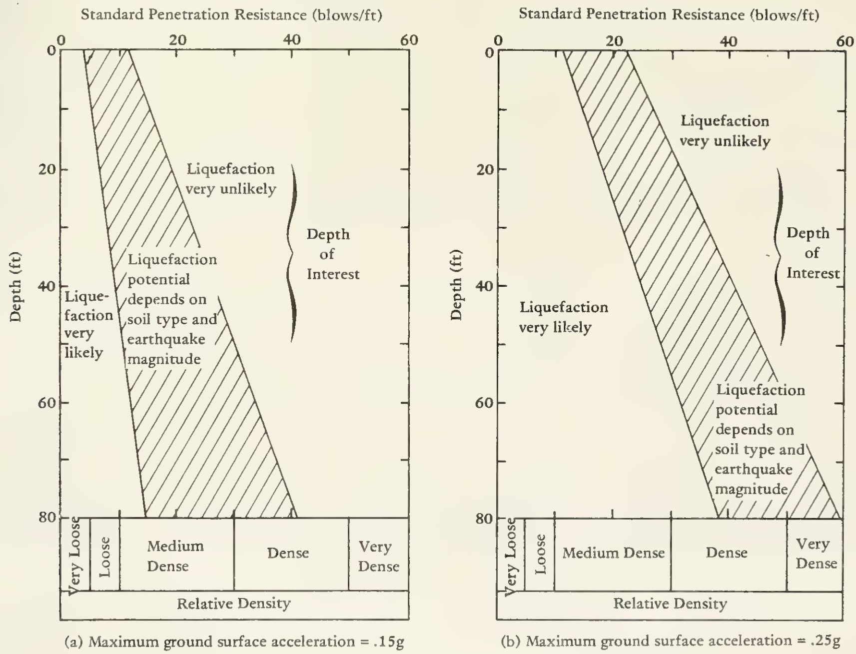


Figure 6-9. Maximum acceleration associated with earthquakes of various magnitudes (from Seed et al., 1969).

major earthquakes. While the shear in the soil may be increased, the effect on anchor capacity is unknown. A granular soil's susceptibility to strength reduction during an earthquake is primarily a function of its descriptive relative density (i.e., loose, dense), and partially a function of soil depth. The criteria for liquefaction are given in Figure 6-10 for two peak acceleration levels. Interpolation or limited extrapolation can be used to assess stability at a site based on the exact value of peak acceleration determined from Figure 6-9. Conditions should be assessed at the depth of the keyed anchor and just above it. If analysis of a site and its expected earthquake indicates

that liquefaction is very likely, the site is hazardous to an extent. In this situation, if the anchor is loaded in any manner during the earthquake (such as with a subsurface buoy) or within a time of about 0.2 t_{cd} (Figure 6-5) immediately following the earthquake, the anchor will likely fail. For soils that will liquefy under earthquake loading, the value of t_{cd} is typically quite short - a matter of minutes at most. Situations that classify as potentially liquefiable are potentially hazardous in the same manner. Factors of safety relative to the anchor load or anchor capacity are meaningless in this type of earthquake loading as the entire soil mass is in a state of failure when liquefaction occurs. For anchors that are loaded for a significant percentage of the time in areas prone to major earthquakes, site conditions which indicate a potential or likelihood for liquefaction should be avoided. For applications having a lower consequence of failure, the possibility (typically a low probability over the lifetime of an embedment anchor system) of a major earthquake in the vicinity and the resultant possiblity of an anchor failure may be acceptable.



(a) Maximum ground surface acceleration = .15g

(b) Maximum ground surface acceleration = .25g

Figure 6-10. Liquefaction potential profiles for earthquake loading of granular soils (from Seed and Idriss, 1971).

7. SAMPLE PROBLEMS

Two sample problems are provided in this section as a guide. One problem is for an anchor in cohesive soil and the other for an anchor in cohesionless soil. Each step in the solution is referenced to the appropriate paragraph of this report.

7.1 COHESIVE SOIL

A CEL 20K propellant-embedded anchor with a 0.9 x 0.9-meter (3 x 3-ft) clay fluke is to be installed 1,100 km (700 miles) east of the Hawaiian Islands at a water depth of 5,500 meters (18,000 ft). It will be used to moor a subsurface buoy about 60 meters (200 ft) below the water surface for a period of several years. The installation procedure will result in a single impulse load of between 0.01 and 0.1 second duration. Cyclic loading will not be applied to the anchor. The remoteness of the site precludes a site survey. What is the maximum allowable lift of the buoy assuming a neutrally buoyant line is used in the mooring? What is the holding capacity for an impulse loading?

7.1.1 Site Properties

Because a site survey cannot be conducted, the procedures of paragraph 2.3 (specifically paragraph 2.3.4(2)) are utilized to estimate soil properties.

The site is in the deep ocean. The calcite compensation depth (CCD) needs to be checked to determine whether the soil is pelagic clay or calcareous ooze. From Figure 2-2, the CCD is found to be 4.5 km

(14,700 ft). The site at 5.5 km (18,000 ft) is below the CCD; therefore, the soil is probably pelagic clay. Typical properties are given by Figure 2-7. A mid-range value is chosen as adequate factors of safety will be included later.

7.1.2 Hazardous/Unusual Conditions

None of the examples in paragraph 2.4 apply. The site does not present any hazardous/unusual conditions.

7.1.3 Penetration

Penetration into cohesive soils is covered by paragraph 3.1. A tabulation of penetration depths for each CEL propellant-embedded anchor for each typical soil profile of paragraph 2.3.4 is given in Table 3-1. From Table 3-1, penetration of the 20K anchor clay fluke in the lower bound and upper bound pelagic clay is found to be 14.3 meters (47 ft) and 11.3 meters (37 ft), respectively. For a mid-range soil profile, D_p is midway between these values, or 12.8 meters (42 ft).

7.1.4 Fluke Keying

Using Equation 4-1 from paragraph 4.1:

$$D = D_p - 2\ L \qquad (4\text{-}1)$$

$$= 12.8 - (2)(0.9)$$

$$= 11\ \text{m}\ (36\ \text{ft})$$

7.1.5 Static Holding Capacity

The short-term static holding capacity in cohesive soil is calculated from Equation 5-1 in paragraph 5.1.1:

$$F_{st} = A\,\bar{N}_c\,s_u\,f(0.84 + 0.16\,B/L) \qquad (5\text{-}1)$$

where s_u = 20.7 kPa (3.0 psi or 432 psf) (Figure 2-7)

f = 0.7 (paragraph 5.1.1)

B = 0.9 m

L = 0.9 m

$A = B \times L = 0.81\ m^2$

$D/B = 11/0.9 = 12$

$\bar{N}_c$ = 15 (Figure 5-1)

Therefore,

$$F_{st} = 0.81(15)(20.7)(0.7)[0.84 + 0.16(0.9/0.9)]$$

$$= 176\ \text{kPa-m}^2 = 176\ \text{kN (39,600 lb)}$$

Because the anchor will be in service for several years under sustained loading from the subsurface buoy, the long-term static capacity must be checked. The long-term capacity is calculated from Equation 5-3 in paragraph 5.2.1. Because the soil is loose (soft), the drained strength parameters are used in Equation 5-3.

$$F_{\ell t} = A\left(c'\,\bar{N}'_c + \gamma_b\,D\,\bar{N}_q\right)(0.84 + 0.16\,B/L) \qquad (5\text{-}3)$$

where $(2/3)c'$ = $(2/3)(3.5\ \text{kPa}) = 2.3\ \text{kPa}\ (48\ \text{lb/ft}^2)$
(Figure 2-7 and paragraph 5.2.1)

$\bar{N}'_c$ = 9 (Figure 5-3)

γ_b = $380\ \text{kg/m}^3\ (24\ \text{lb/ft}^3)$ (Figure 2-7)

$\tan^{-1}(2/3\tan\phi)$ = $\tan^{-1}(2/3\tan 35°) = 25$ degrees
(Figure 2-7 and paragraph 5.2.1)

$\bar{N}_q$ = 6 (Figure 5-2)

Therefore,

$$F_{\ell t} = 0.81[(2.3)(9) + (380)(11)(6)][0.84 + 0.16(0.9/0.9)]$$

$$= 0.81\ (20.7 + 25{,}080)(0.0098) = 17\ \text{kN} + 199\ \text{kN}$$

$$= 216\ \text{kN}\ (48{,}600\ \text{lb})$$

The short-term holding capacity is the governing static condition since it is less than the long-term static holding capacity. Applying the factor of safety (3) recommended (paragraph 5.1) when properties are not well known, the allowable static load is found to be:

Allowable static load = 176/3 = 58.7 kN (13,200 lb)

7.1.6 Impulse Loading Holding Capacity

The impulse loading holding capacity is determined from Equation 6-1 in paragraph 6.1.1.

$$F_I = F_{st}\ I\ R_c\ R_I\ I_f \qquad (6\text{-}1)$$

where F_{st} = 176 kN (40,600 lb)

I = 2 from Figure 6-1 for a load duration between 0.01 and 0.1 second and a normally consolidated, moderately sensitive clay

R_c = 1 (paragraph 6.1.1)

$$R_I = 1.33\ e^{-1.15\ \bar{f}} \qquad (6\text{-}2)$$

$\bar{f}$ = average frequency, impulses per hour, over a 4-hour span

$R_I = 1.33\ e^{-1.15(0.25)} = 1.0$

I_f = not used because impulse duration >0.01 second

Therefore,

$$F_I = 176(2)(1)(1) = 352\ \text{kN}\ (79{,}100\ \text{lb})$$

Using the recommendations for factor of safety (same as for static conditions)

Allowable impulse load = 352/3 = 117 kN (26,300 lb)

7.1.7 Solution Summary

The allowable short-term static holding capacity is less than the allowable long-term static holding capacity and is, therefore, the governing condition. The allowable load is also the maximum allowable lift of the subsurface buoy.

Allowable static load = 58.7 kN (13,200 lb)

The allowable impulse load for a duration of between 0.01 and 0.1 second is

Allowable impulse load = 117 kN (26,300 lb)

7.2 COHESIONLESS SOIL

A CEL 100K propellant-embedded anchor is to be installed in a sand seafloor to moor a large surface buoy. The 0.75 x 1.5-meter (2.5 x 5-ft) sand fluke will be used. The sand has been thoroughly tested and found to be well-graded, have an angle of internal friction of 35 degrees, be of medium density (γ_b = 880 kg/m^3; 55 pcf), and have a permeability of 1 x 10^{-3} cm/sec (3 x 10^{-5} ft/sec). The anchor is scheduled to be used for 2 years. Impulse loading is not expected, but passing storms will generate cyclic load conditions. The worst storm is expected to create 30,000 loading cycles at about 8-second intervals. In addition, each of 15 smaller storms each year is expected to create about 20,000 loading cycles. The site is located about 40 km (25 miles) from an active earthquake fault zone with a maximum expected earthquake of magnitude 7. To assist the mooring designer, determine the static holding capacity and provide analysis of the dynamic loading limits or effects.

7.2.1 Site Properties

The site properties are given as:

Internal friction angle	=	35 degrees
Unit buoyant weight	=	880 kg/m^3 (55 pcf)
Permeability	=	1×10^{-3} cm/sec (3×10^{-5} ft/sec)

7.2.2 Hazardous/Unusual Conditions

Since none of the examples of paragraph 2.4 seem to apply, it is assumed that there are no hazardous/unusual conditions.

7.2.3 Penetration

Penetration into cohesionless soils is covered by paragraph 3.2. For the CEL propellant-embedded anchors, a tabulation of embedment depths is given for a range of sand densities. From Table 3-2, D_p for a medium dense sand is found to be about 7 meters (23 ft).

7.2.4 Fluke Keying

Using Equation 4-2 from paragraph 4.2

$$D = D_p - 1.5\ L \tag{4-2}$$

$$= 7 - (1.5)(1.5)$$

$$= 4.8\ \text{m}\ (15.7\ \text{ft})$$

7.2.5 Static Holding Capacity

Short-term static holding capacity is calculated using Equation 5-2 from paragraph 5.1.2.

$$F_{st} = A\, \gamma_b\, D\, \bar{N}_q (0.84 + 0.16\ B/L) \qquad (5\text{-}2)$$

where

$B = 0.75$ m (2.5 ft)

$L = 1.5$ m (5 ft)

$A = B \times L = 1.13$ m³ (12.2 ft²)

$\gamma_b = 880$ kg/m³ (55 lb/ft²)

$D = 4.8$ m (15.7 ft)

$D/B = 6.3$

$\bar{N}_q = 18$ (Figure 5-2)

Therefore,

$$F_{st} = 1.13(880)(4.8)(18)[0.84 + 0.16(0.75/1.5)]$$

$$= 775 \text{ kN } (174{,}000 \text{ lb})$$

No distinction is made between long- and short-term static capacity in sand; therefore:

$$F_{lt} = 775 \text{ kN } (174{,}000 \text{ lb})$$

Applying the factor of safety recommended when soil properties are well known,

Allowable static load = 775/2 = 388 kN (87,000 lb)

7.2.6 Cyclic Loading

7.2.6.1 Strength Loss During Cylic Loading. Following the procedures of paragraph 6.2.1, t_{cd} is found from Figure 6-5 to be about 50 seconds for the permeability and anchor width given. With an 8-second period, about seven cycles are applied in the period t_{cd}.

Entering Figure 6-6 with seven load cycles and using the curve for medium density soils, the double amplitude cyclic load is found to be about 43% of static capacity or 333 kN (43% of 775 kN) (74,900 lb). This limit applies when the quasi-static load does not exceed one-third of the static capacity or 258 kN (775 kN/3) (58,000 lb). Applying a factor of safety of 1.25 to the determined double amplitude cyclic and the quasi-static loads as recommended in paragraph 6.2, the allowable double amplitude cyclic component is 266 kN (333/1.25) (59,900 lb) on a 206-kN (258/1.25) (46,400-lb) quasi-static load.

7.2.6.2 Cyclic Creep. Cyclic creep is determined from paragraph 6.2.2 and Figure 6-7. Over the two-year life of the anchor, 630,000 loading cycles (n_T = 30,000 + (15)(2)(20,000)) are expected. From Figure 6-7 for denser granular soils, the double amplitude cyclic load allowed is found to be 28% of the allowable short-term holding capacity or 217 kN (28% of 775 kN) (48,800 lb). The value applies when the quasi-static load component is less than 20% of the static holding capacity or 155 kN (20% of 775 kN) (34,800 lb). Applying the recommended factor of safety (1.25; paragraph 6.2) to the double amplitude cyclic and quasi-static loads, the allowable loads for creep are a 174-kN (217/1.25) (39,000-lb) cyclic component on a 124-kN (155/1.25) (27,900-lb) quasi-static load.

7.2.6.3 Earthquake Loading. Procedures for determining the effects of earthquake loading are given in paragraph 6.3. Using Figure 6-9, the maximum acceleration at the site is found to be about 0.15g for the 40-km (25-mile) earthquake fault distance and magnitude 7.5 earthquake given. Next, Figure 6-10 is entered using this acceleration, the depth of the anchor (4.8 m; 15.7 ft), and the relative soil density (medium dense). Using Figure 6-10a for 0.15g ground acceleration, it is noted that for the soil and depth given, liquefaction potential borders between very unlikely and possible.

7.2.7 Solution Summary

Allowable static load	=	388 kN (87,000 lb)
Allowable cyclic loading for an 8-second loading period	=	266-kN (59,900-lb) cyclic component on a 206-kN (46,400-lb) quasi-static load
Allowable cyclic creep loading for 630,000 cycles	=	174-kN (39,000-lb) cyclic component on a 124-kN (27,900-lb) quasi-static load
Earthquake susceptibility	=	liquefaction potential borders between very unlikely and possible

8. ACKNOWLEDGMENTS

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COMCBPAC Operations Off, Makalapa HI
COMDEVGRUONE San Diego, CA
COMFLEACT, OKINAWA PWO, Kadena, Okinawa
COMNAVBEACHPHIBREFTRAGRU ONE San Diego CA
COMNAVMARIANAS Code N4, Guam
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COMSUBDEVGRUONE Operations Offr, San Diego, CA
NAVSURFLANT Norfolk, VA
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DLSIE Army Logistics Mgt Center, Fort Lee, VA
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DOD Explosives Safety Board (Library), Washington DC
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DTIC Defense Technical Info Ctr/Alexandria, VA
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LIST OF SYMBOLS

A	Projected fluke area
A_f	Penetrator frontal area
A_s	Penetrator side area
B	Fluke width or diameter
c	Soil cohesion intercept
c'	Soil cohesion to be used for loose cohesive soils
C_D	Inertial drag coefficient
$C_{\dot{e}}$	Empirical soil strain rate factor
D	Depth of embedment of fluke
D_p	Penetration depth
f	Correction factor to account for soil disturbance
$\bar{f}$	Number of impulses in a specified time frame
F_I	Impulse loading holding capacity
$F_{\ell t}$	Long-term static holding capacity
F_{st}	Short-term static holding capacity
F_1	Inertial drag force
F_2	Bearing component force
F_3	Side adhesion force
h	Depth of earthquake epicenter
i	Subscript for the i^{th} increment
I	Influence factor for adjusting soil strength
I_f	Inertial factor to use for capacity increase for very rapid loading
L	Fluke length or diameter
M	Penetrator mass
M'	Penetrator effective mass
n_C	Number of loading cycles
n_T	Total number of loading cycles in an anchor's lifetime
$\bar{N}_c$	Short-term holding capacity factor in cohesive soil
$\bar{N}'_c$	Long-term holding capacity factor in cohesive soil
$\bar{N}_q$	Holding capacity factors for drained or frictional condition
$\bar{N}_{qI}$	Cohesionless soil holding capacity factor adjusted for impulse loading
R_c	Reduction factor to use for determining cyclic loading holding capacity
R_I	Reduction factor to use for repeated impulses
s_u	Soil undrained shear strength
$S_e{}^*$	Empirical maximum soil strain rate
S_t	Soil sensitivity
t	Penetrator thickness
t_{cd}	Time required for dissipation/ redistribution of most excess pore water pressure in anchor fluke vicinity
v	Penetrator velocity
V	Penetrator volume
W	Penetrator weight
W_b	Penetrator buoyant weight
z	Depth in soil
α	Half-angle of the penetrator tip
γ_t	Soil total unit weight
γ_b	Soil buoyant unit weight
δ^*	Side adhesion factor
ρ	Soil mass density
ϕ	Soil internal friction angle
ϕ_I	Soil impulse adjusted internal friction angle